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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

FLOW AROUND BENDS IN STABLE CHANNELS

By C. A. Mockmore,[1] M. Am. Soc. C. E.

Synopsis

The purpose of the project discussed herein was to make careful flow measurements under laboratory conditions, to record, and to study the phenomenon of flow around a stable curved channel.

An open channel, 18 in. wide and 10 in. deep, was built of transparent pyralin sections so they could be fitted together in combinations of tangents and curves. Motion pictures were taken of the flow phenomena as indicated by objects floated on the water surface, dragged or rolled on the stream bed, or held in suspension. The latter was accomplished by use of drops of a mixture of n. Butyl Plithalate and technical Xylol, colored with a deep red oil dye and having the same specific gravity as that of water. The water velocities were measured at various cross sections from which velocity contours were prepared.

Two different sets of assumptions were made as to the nature of the filamental velocities of flow around a bend, and a mathematical analysis was made of the streamlines, accelerations, and velocity components by use of the laws of hydrodynamics.

The Laboratory Experiments

Complications are inherent with the flow of water in any channel where the particles of water are forced to undergo a general change in direction. If there were streamline flow of equal filament velocities at the entrance to the bend and if friction could be eliminated so the water would act as a perfect fluid, no spiral motion could develop. The two principal causes of spiral flow in either closed or open channels are:

(*a*) The existence of friction on the channel walls, resulting in higher filamental velocities near the center of the channel than near the walls; and

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Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **August 1, 1943.** The author of this paper is a cooperating member of the Committee on Hydraulic Research. He accepted this subject as a research project at the request of the Committee and submits the results of his studies in this paper.

[1] Prof. and Head, Dept. of Civ. Eng., Oregon State Coll., Corvallis, Ore.

(*b*) The centrifugal force required to deflect the particles of water from straight-line motion.

The existence of a condition of spiral flow around bends in open channels was first enunciated by Prof. James Thompson[2] of England in 1876.

A study of the flow conditions in the bend of a natural stream would involve many complexities, not only because of the difficulty in obtaining accurate measurements of the filamental velocities at specified points in a large stream, but also because of the many irregularities in, and the instability of, the stream bed. The laboratory is the most logical place, therefore, to study such fundamentals.

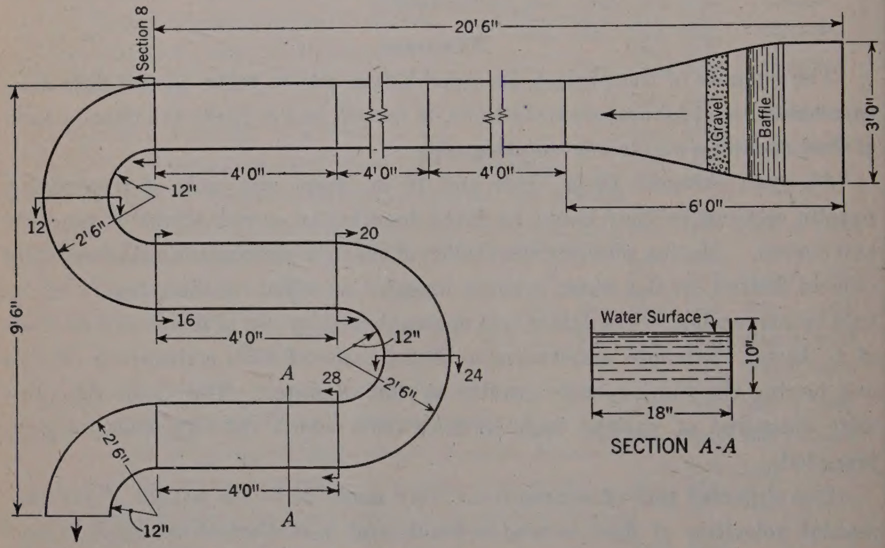

FIG. 1.—EXPERIMENTAL APPARATUS FOR FLOW OF WATER AROUND BENDS IN OPEN CHANNELS

An open channel, 18 in. wide and 10 in. deep, was constructed in sections as shown in Fig. 1. The bend sections and one 4-ft tangent were made of transparent pyralin to facilitate observation of the flow and for the approximation of the desired rate. The final rate of flow was obtained by use of a carefully calibrated concrete volumetric measuring tank. The filamental velocities were measured with a special midget current meter and checked with a pitot tube, the latter being one used in numerous other experiments, which had been carefully calibrated.[3,4,5]

The channel was set on a slope of 0.005% and the depth of the water was controlled by a tail gate at the outlet of the channel. In some experiments the gate caused a backwater curve which gave a different depth of water at each section of the channel, but this was not of enough consequence to require its elimination.

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[2] "On the Origin and Winding of Rivers in Alluvial Plains, with Remarks on the Flow Around Bends in Pipes," by James Thompson, *Proceedings*, Royal Soc. of London, March, 1876, pp. 5–8.

[3] "Flow Characteristics in Elbow Draft Tubes," by C. A. Mockmore, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 412.

[4] "Hydraulic Laboratory Practice," J. R. Freeman, Editor, A.S.M.E., 1929, p. 712.

[5] "Flow of Water Around Bends in Pipes," by D. L. Yarnell and F. A. Nagler, *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1020.

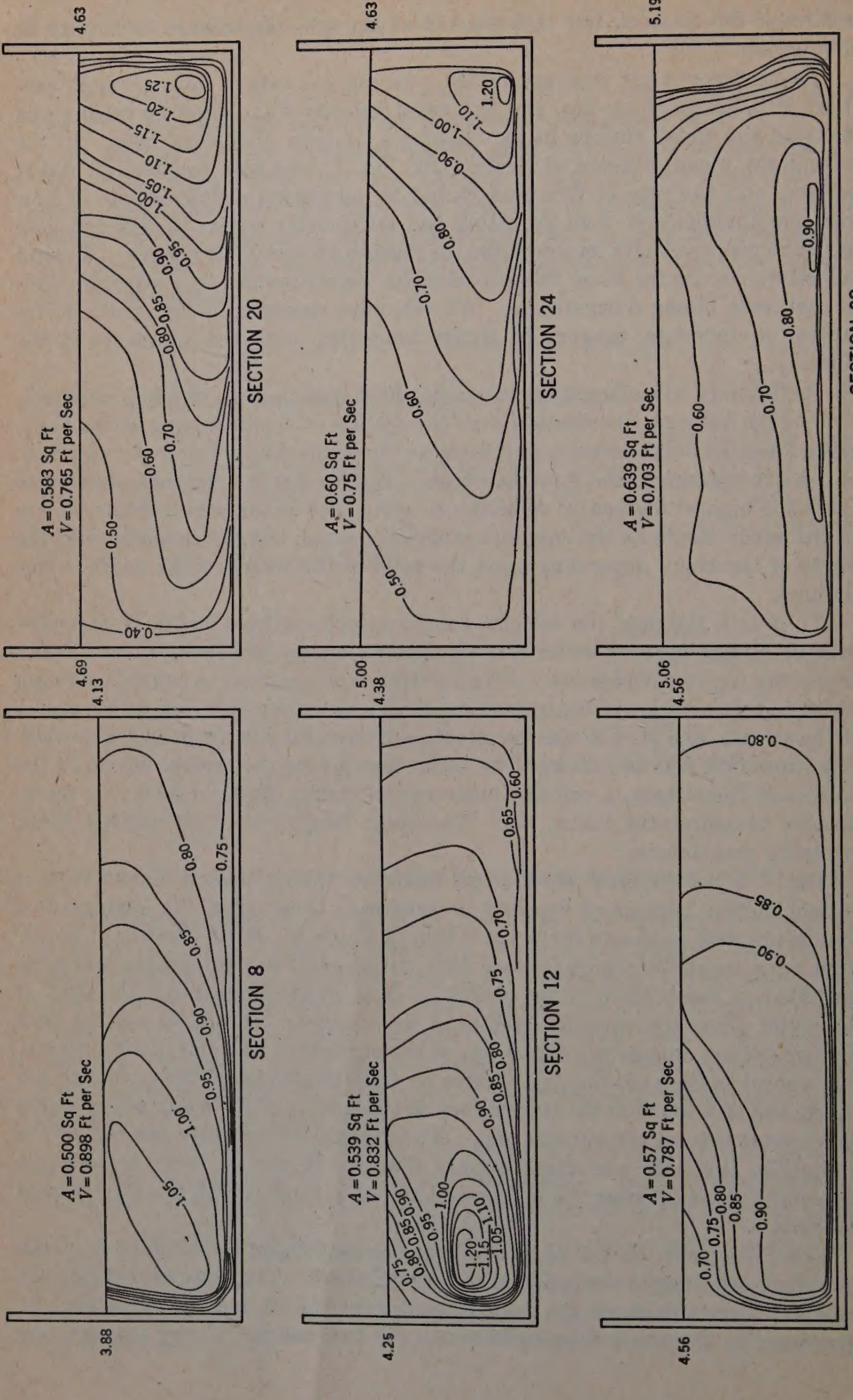
Two series of tests were conducted. In one the rate of flow was held constant at 0.449 cu ft per sec, and the mean velocity was varied by raising and lowering the water surface in the channel by means of the tail gate. In the second the mean velocity at Section 16, Fig. 1, was held constant at 0.5 ft per sec, and the rate of flow was varied by admitting a desired rate of flow into the channel and then adjusting the micrometer screws on the tail gate until the mean velocity as computed for Section 16 was 0.5 ft per sec. Typical velocity contours for these two experiments are shown in Fig. 2 with the view in each case facing downstream. All velocities recorded in Fig. 2 are downstream components, tangent to circles concentric with the curvature of the bend.

Preliminary to each test considerable effort was made to obtain a comparatively even velocity distribution over the section at the beginning of the bend, using various kinds of screens and floats at the channel inlet below the weir box; but in every stage of the flow the effect of the bend was reflected upstream to cause the highest filamental velocities to occur, not at the center of the stream as ordinarily would be the case in a straight channel, but somewhat toward the inside of the bend, depending upon the ratio of the width to the depth of the channel.

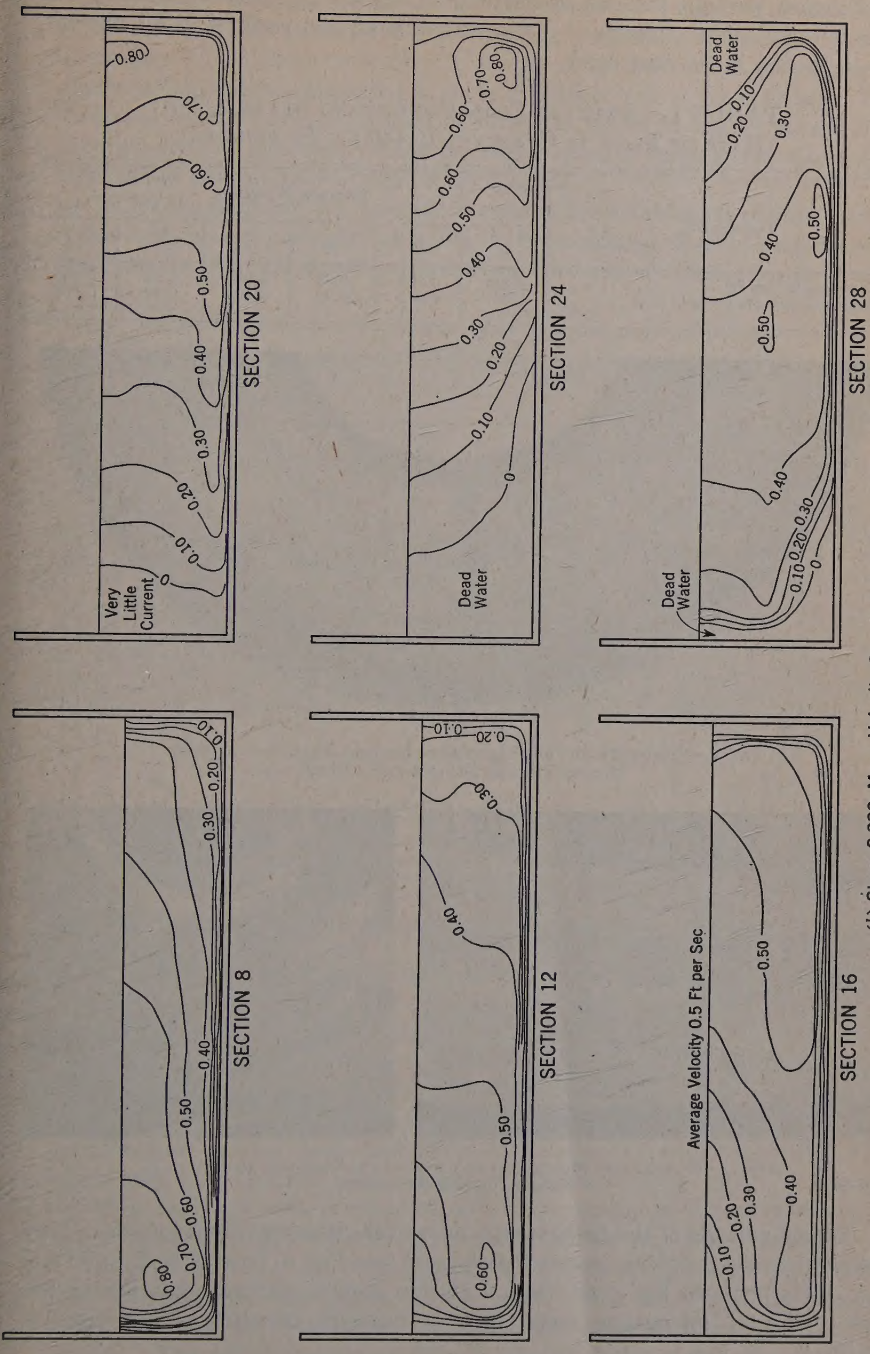
To obtain the data for velocity contours such as those in Fig. 2, the pitot tube or midget current meter was placed in a clamp and fastened to a beam across the top of the channel. Five verticals were selected as suitable through which to raise and lower the meter for velocity measurements, one at the center of the stream, one at each quarter point, and two at 0.5 in. from each side-wall. The procedure was as follows: The meter was lowered to within 0.5 in. of the bottom of the stream, a velocity measurement taken, then raised 0.5 in. more, another measurement taken, etc. The stage heights were all referred to an arbitrary zero datum.

Fig. 3 is a composite photograph made by taking several frames from a 16-mm motion picture of the first or upstream bend when the average flow conditions were as shown for Stage 0.600, in Table 1. Some crystals of potassium permanganate, which formed dark patches in the photograph, had been sprinkled on the bottom of the channel. The broken lines follow the path of the water along the extreme bottom of the channel. It will be evident that the streamlines crossed the center line of the channel at an angle of 45° or more. The unbroken lines are the paths taken by confetti strewn on the surface of the water, and indicate that the streamlines on the surface of the water edged across the channel toward the outside wall. The average downstream velocity in the channel in this case was slightly more than 0.5 ft per sec, with the highest filamental velocities near the inside wall and about one fourth the way around the bend.

Confetti strewn on the water surface seemed to edge to the outside of the first bend, but hugged the inside of the second bend. The reason for this is not apparent from a study of the velocity contours (Fig. 2), but the cause may be attributed to the short tangent between the two bends. If the tangent had



(*a*) Stage 0.406; Rate of Flow Constant (0.449 Cu Ft per Sec)



(b) Stage 0.030; Mean Velocity Constant (0.5 Ft per Sec)

FIG. 2.—TYPICAL VELOCITY CONTOURS

been longer, perhaps the surface currents of the second bend would have had time to overcome the effects of the upstream bend and perform as did the surface velocities in the first bend.

TABLE 1.—AVERAGE FLOW CONDITIONS, STAGE 0.600; RATE OF FLOW IS CONSTANT (0.449 CU FT PER SEC)

Description	SECTION NUMBER:					
	8	12	16	20	24	28
Area of section (sq ft)	0.811	0.843	0.88	0.90	0.91	0.963
Average velocity (ft per sec)	0.61	0.532	0.51	0.494	0.49	0.466

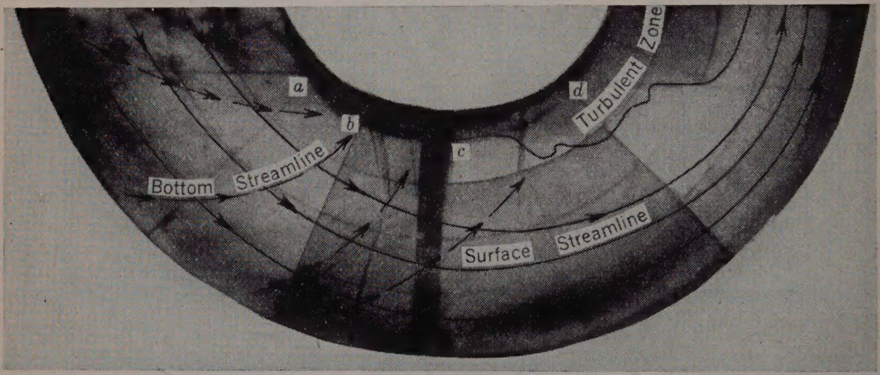


FIG. 3.—COMPOSITE PICTURE MADE FROM SEVERAL FRAMES OF A 16-MM MOTION PICTURE FLOW AROUND BEND

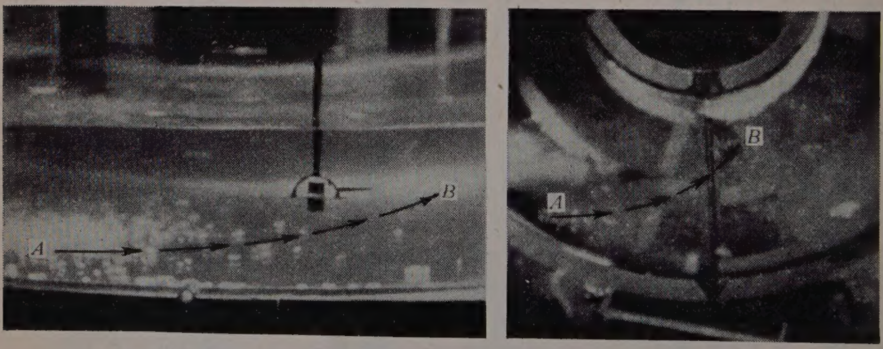
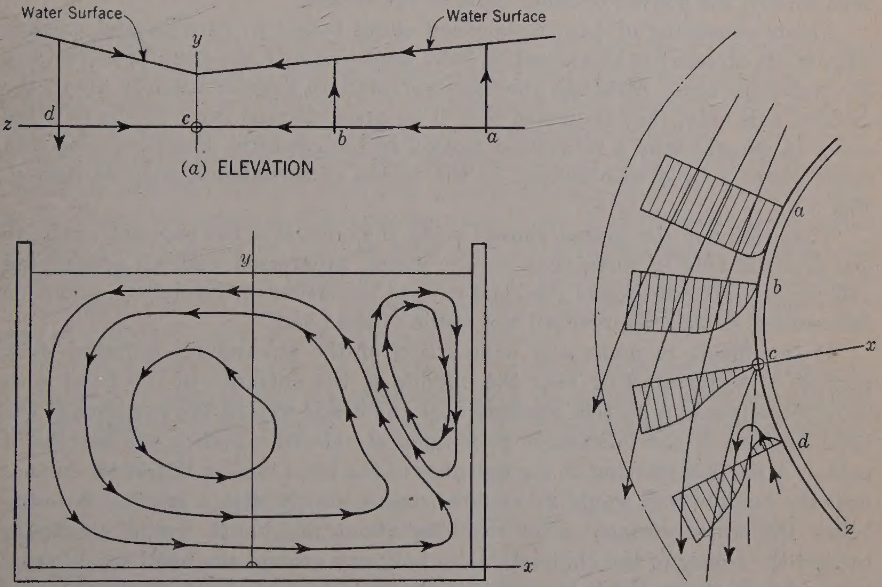


FIG. 4.—RICE GRAINS MOVED RAPIDLY ACROSS THE CHANNEL FROM A TO B, WHERE A BAR WAS FORMED

The magnitude of the bottom streamline velocities was considerable. Rice grains were swept quickly across the channel (see Fig. 4) from A to B to form a bar. The views in Fig. 4 are from a 16-mm motion picture taken during the tests. The midget current meter used to measure velocities of the water is shown.

On the convex bank in the last half of both bends there existed (see Figs. 2, 3, and 4) regions of unsteady flow in which, at times, the water actually moved upstream. At slightly more than half way around the bend and above mid-depth the streamlines appeared to leave the inside wall of the channel. This is indicated by the darkened region of turbulent water in Fig. 3, caused by the accumulation of dissolved potassium permanganate being swept along by the streamlines nearest the bottom of the channel.

If, for the moment, the velocity component of a thin layer of fluid along the water surface in the vicinity of a, Fig. 5, is considered, it will be apparent that particles not only are moving downstream but also are edging toward the outside of the bend. Then as the particles pass on to c they experience a kind of diverging channel effect, and at d the streamlines diverge from the inside wall causing a backflow somewhat as shown. This effect was admirably demonstrated by filling the channel with still water, liberally covering the stream bed with water-soaked sawdust and suddenly opening the tail gate wide. As the water accelerated, spiral flow developed, and the finely divided sawdust particles were carried along the bottom of the channel to the region of d, Fig. 5, where they were churned about violently in the turbulent eddy. The sawdust was then replaced with crystals of potassium permanganate, and the colored water followed the same pattern as the transported sawdust (Fig. 3).



(*a*) ELEVATION

(*b*) SECTION AT *d*

(*c*) PLAN

Fig. 5.—Development of Turbulence at Inside of Bend

In the lower half of the section at d, Fig. 5, the water particles had a velocity component toward the inside of the bend. This prevented the region of turbulence from developing to the same extent as it did nearer the water surface. The velocity contours (Fig. 2) give evidence of this phenomenon.

To gain information on the nature of the streamlines around the bend, use was made of a mixture of n. Butyl Plithalate and technical Xylol colored with a deep red oil dye and having the same specific gravity as water. The mixture was admitted into the stream through capillary tubes, and it came from the tubes in globules about the size of a small pea, providing a ready means of tracing the streamlines. Motion pictures were taken giving an excellent record of the existence, extent, and nature of the spiral flow. When the globules passed the section half way around the bend and happened to be along the bottom of the channel and midway between the side-walls, their path crossed the center line of the channel at an angle of about 45°, indicating a strong current toward the convex or inner bank of the stream.

Floats consisting of ⅜-in. hedgewood cubes boiled in paraffin and marked with white crosses for identification were used to study the angular velocity of the water surface. Although the blocks showed an angular velocity about the vertical axis only, they indicated that if an open channel turns to the left, the cubes in general take a rotational motion in the opposite direction, except in cases where a cube was trapped in the region of turbulence, such as near d, Fig. 5.

The motion of rice grains, shoved along the bottom of the channel, or chemical globules held in suspension by the water, progressed with an accelerated velocity from the outside of the bend toward the center of the channel and then decelerated as they approached the inside of the bend.

It is difficult to make any comparison of the streamlines followed by a particle of water starting near the outside at the entrance of the bend with the streamlines of a particle starting near the inside wall at the entrance of the bend because of the differences in filamental velocities and in the lengths of path. A particle starting at the entrance of the bend and on the water surface near the outside wall would move downstream slowly with a gradual lowering below the water surface; after reaching about mid-depth would accelerate toward the center of the channel; when half way around the bend would cross the center line near the bottom of the channel at an angle of nearly 45°; and would end in the turbulent region. A particle starting at the entrance of the bend and on the water surface near the inside wall would move swiftly downstream with a strong velocity component outward, upon approaching the outside wall would gradually be lowered below the water surface, and would leave the bend near the bottom of the channel. On the average the liquid completed half a turn of the spiral in flowing around the 180° bend. However, the time required for the different particles to move through their allotted courses differed considerably, the shorter period being for those starting near the inside wall.

The transverse profile of the water surface at Section 12, Stage 0.406 (Fig. 2(*a*)), shows that the water is higher at the outside wall than at the inside wall. This condition, due to the centrifugal force of the water flowing around the bend, is not enough in itself to induce the spiral flow, but it is one of the two essential conditions, the other being a difference between the filamental velocities of the water near the surface and near the bottom of the channel.

NOMENCLATURE

The letter symbols used in this paper are defined where they first appear and are assembled for convenience of reference in the Appendix.

THEORY OF SPIRAL FLOW

The theory of the induced spiral has been discussed admirably by Sherman M. Woodward,[6] Hon. M. Am. Soc. C. E., whose reasoning may be explained by reference to Fig. 6. If the layer of water E–F moves in a curve, a centrifugal force of

$$F = \frac{m\,V^2}{r} \qquad (1)$$

must be directed toward the center of curvature. In Eq. 1, m is the mass of the water; V, the velocity in feet per second; and r, the radius of curvature. The excess of pressure on the face D–H over that on C–G causes the water to be deflected toward the center of curvature on the side toward A. This excess of pressure can exist only when the water surface at D is higher than at C. The layer near the bottom G–H has a lower velocity than the central layer E–F. Since the pressure difference between F and E is the same as that between G and H, it follows that the layer G–H has a shorter radius of curvature than the layer E–F. A spiral flow inevitably results, in this case clockwise when facing downstream. If the velocity of the layer E–F were lower than that of G–H, the spiral would be counterclockwise.

Since the vertical curve of the velocity variation approximates a parabola in shape, it is natural that the crosscurrents of the spiral be most pronounced near the bottom of the stream. At Section 12, Stage 0.406, the water surface at the outside and inside of the bend had a difference of elevation of 0.13 in. This is sufficient to induce toward the inside of the bend a theoretical velocity of 0.8 ft per sec, which is nearly as large as the average downstream velocity in the section as a whole. Friction losses greatly reduce the theoretical velocity,

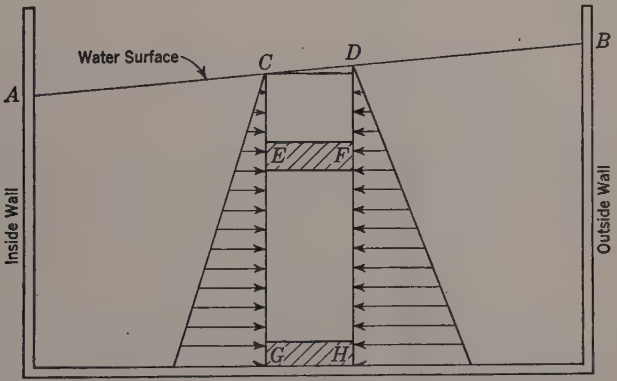


FIG. 6.—CROSS SECTION OF A BEND OF OPEN CHANNEL, FACING DOWNSTREAM

but the motion pictures taken verify the existence of a crosscurrent throughout the entire 180° bend, with the most pronounced crosscurrents near the very bottom of the stream.

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[6] "Hydraulics of the Miami Flood Control Project," by S. M. Woodward, *Technical Reports*, Miami Conservancy Dist., Dayton, Ohio, Pt. VII, 1920, p. 264.

If the channel under consideration happens to be rather wide and comparatively shallow, if the water is flowing through a bend, such as may exist in a natural river or model of a river, and if it encounters channel irregularities, there undoubtedly will be localized turbulence which may tend to distort the spiral flow pattern. The localized turbulence and eddy currents in such a channel may so obscure the spiral flow pattern as to cause doubt in the minds of some as to possibility of its existence in the bend of any open channel.[7] However, if the essential driving forces exist at a bend in the stream, with higher downstream velocities near the water surface than near the stream bed, a general spiral motion as first enunciated by Thompson[2] is inevitable. A careful weighing of the basic cause of the spiral will usually suffice to convince the skeptic.[8]

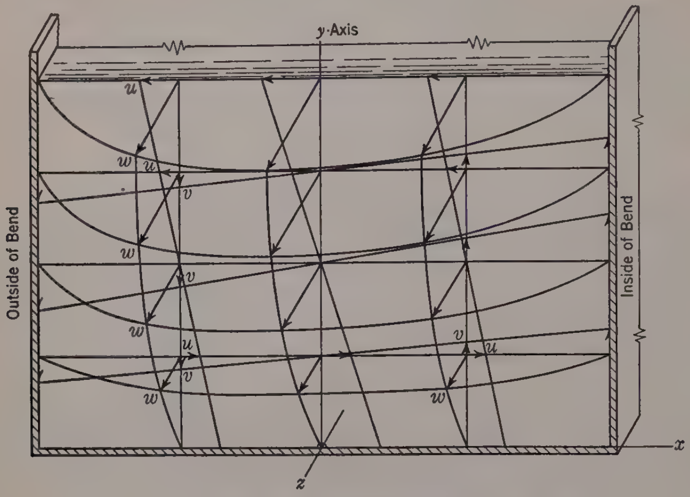

Fig. 7.—Channel Section Showing Solid of Flow and Assumed Velocity Components

Mathematical Analysis

Suppose an open channel of rectangular section be considered, and that assumptions are made regarding the u-component, v-component, and w-component of the velocity parallel to the x-axis, y-axis, and z-axis as shown in Fig. 7 for a frictionless fluid as follows:

1. Parabolic variation of the u-component of the velocity, starting with zero at each side-wall and becoming a maximum on the y-axis midway between the two side-walls. Straight-line variation of the u-component, starting with

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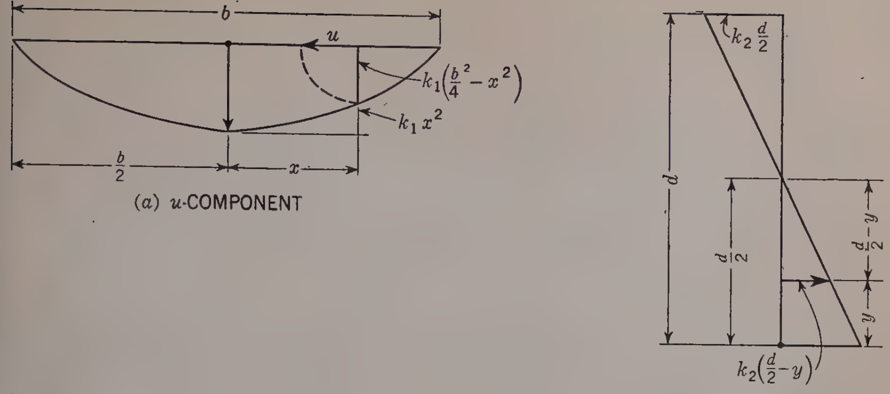
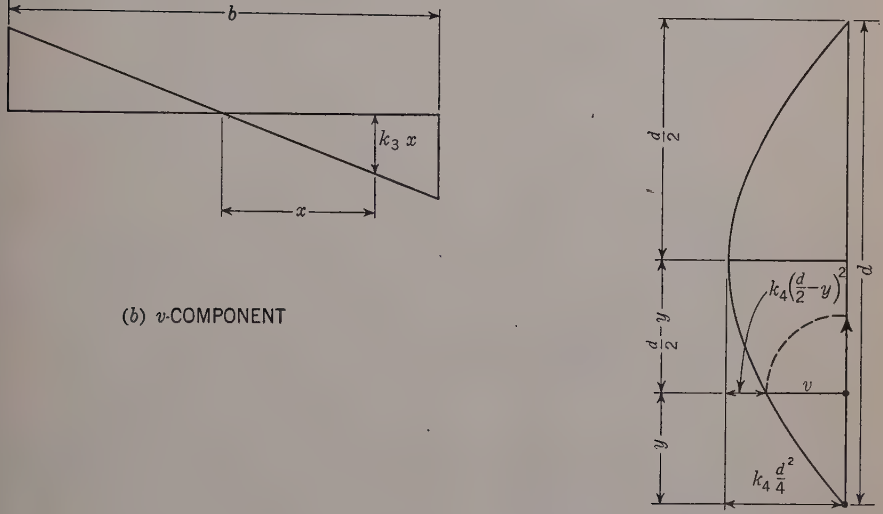
[7] "Flow in River Bends," by Herbert D. Vogel and Paul W. Thompson, *Civil Engineering*, May, 1933, pp. 266–268.

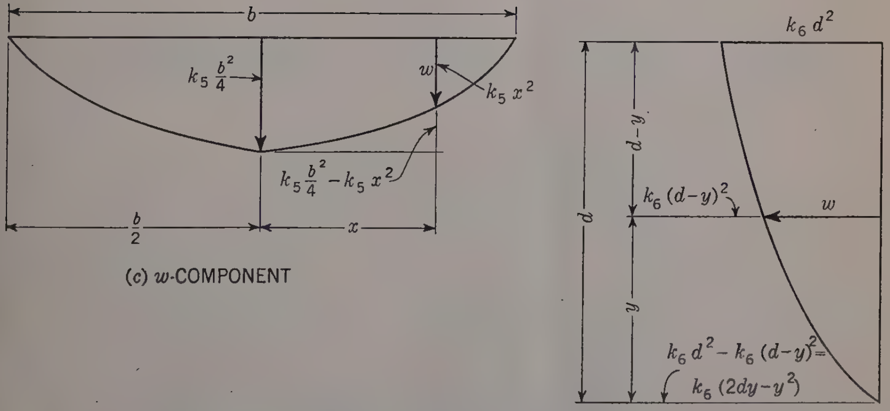
[8] "Existence of Helicoidal Flow," by Herbert D. Vogel and Paul W. Thompson, *ibid.*, July, 1934, pp. 370–371.



(a) u-COMPONENT



(b) v-COMPONENT



(c) w-COMPONENT

FIG. 8.—VELOCITY COMPONENTS

zero at mid-depth of the channel and becoming a maximum at top and bottom of channel, with opposite signs.

2. Straight-line variation of the v-component of the velocity, starting with zero on the y-axis and becoming a maximum at each side-wall, with opposite signs. Parabolic variation of the v-component starting with zero at top and bottom of channel, with a maximum value at mid-depth.

3. Parabolic variation of w-component of the velocity, starting with zero at the bottom of the channel and becoming a maximum at the water surface. Parabolic variation of w-component, starting with zero at each side-wall and becoming a maximum on the y-axis midway between the two side-walls.

Then, if b and d, respectively, represent the breadth of channel and depth of water, the velocity components at a particular cross section of the channel may be written in terms of x, y, b, d, and certain coefficients, which in this discussion will be called k_u, k_v, and k_w, for the u-component, v-component, and w-component of the velocity. Referring to Fig. 8,

$$u = k_1\,k_2\left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right) = k_u\left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right) \quad\text{.....(2a)}$$

$$v = k_3\,x\left[k_4\frac{d^2}{4} - k_4\left(\frac{d}{2} - y\right)^2\right] = k_3\,k_4\,x\,(y\,d - y^2) = k_v\,x\,(y\,d - y^2) \quad\text{..(2b)}$$

and

$$w = k_5\left(\frac{b^2}{4} - x^2\right)k_6\,(2\,y\,d - y^2) = k_w\left(\frac{b^2}{4} - x^2\right)(2\,y\,d - y^2) \quad\text{...(2c)}$$

In Eqs. 2, k_u and k_v have dimensions of $L^{-2}\,T^{-1}$, and k_w has dimensions of $L^{-3}\,T^{-1}$. Assuming for simplicity that $k_u = k_v = k_w = 1$, Eqs. 2 become

$$u = \left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right) \quad\text{.....(3a)}$$

$$v = x\,(y\,d - y^2) \quad\text{.....(3b)}$$

and

$$w = \left(\frac{b^2}{4} - x^2\right)(2\,y\,d - y^2) \quad\text{.....(3c)}$$

Streamlines.—If streamlines are defined as curves the tangents to which everywhere give the direction in which the fluid is moving, then the equation of the streamlines is[9] $\frac{dx}{u} = \frac{dy}{v} = \frac{dz}{w}$, and

$$\frac{dx}{\left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right)} = \frac{dy}{x\,(y\,d - y^2)} \quad\text{.....(4a)}$$

Separating terms and integrating,

$$\log\left(\frac{b^2}{4} - x^2\right) = -\log\,(y\,d - y^2) + C_1 \quad\text{.....(4b)}$$

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[9] "An Introduction to Mathematical Physics," by R. A. Houstoun, Longmans, Green, 1925, Chapter II.

or

$$\left(\frac{b^2}{4} - x^2\right)(y\,d - y^2) = C \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Eq. 5 should be the formula of the streamlines as projected on the x–y plane, which is the plane of the channel cross section. For the y–z plane,

$$\frac{dz}{\left(\dfrac{b^2}{4} - x^2\right)(2\,y\,d - y^2)} = \frac{dy}{x\,(y\,d - y^2)} \quad \ldots\ldots\ldots\ldots (6a)$$

but from Eq. 5,

$$x = \left(\frac{b^2}{4} - \frac{C}{y\,d - y^2}\right)^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6b)$$

then

$$\frac{dz}{\left(\dfrac{C}{y\,d - y^2}\right)(2\,y\,d - y^2)} = \frac{dy}{\left(\dfrac{b^2}{4} - \dfrac{C}{y\,d - y^2}\right)^{0.5}(y\,d - y^2)} \quad \ldots\ldots (6c)$$

and

$$z = \int \frac{C\,(2\,y\,d - y^2)\,dy}{\left(\dfrac{b^2}{4} - \dfrac{C}{y\,d - y^2}\right)^{0.5}(y\,d - y^2)^2} + C_2 \quad \ldots\ldots\ldots\ldots (7a)$$

If the coefficients k_v and k_w are retained, and Eq. $7a$ is made a definite integral,

$$z = \frac{k_w}{k_v}\int \frac{C\,(2\,y\,d - y^2)\,dy}{\left(\dfrac{b^2}{4} - \dfrac{C}{y\,d - y^2}\right)^{0.5}(y\,d - y^2)^2} \quad \ldots\ldots\ldots\ldots (7b)$$

Eq. $7b$ may be evaluated as follows:
First step: Simple reductions put the integral in the form:

$$z = \frac{k_w\,C}{k_v}\int \frac{(2\,d - y)\,dy}{(d - y)\sqrt{(y\,d - y^2)\left[\dfrac{b^2}{4}(y\,d - y^2) - C\right]}} \quad \ldots\ldots (8a)$$

Second step: Let $y = \left(\dfrac{d}{2}\right)(1 + s)$; $k^2 = 1 - \left(\dfrac{16\,C}{b^2\,d^2}\right)$; and $M = \dfrac{4\,k_w\,C}{k_v\,b\,d}$.

Then

$$z = M\int \frac{(3 - s)\,ds}{(1 - s)\sqrt{(1 - s^2)(k^2 - s^2)}} \quad \ldots\ldots\ldots\ldots (8b)$$

Third step: Break up the integrand in three terms as follows:

$$\frac{3 - s}{(1 - s)\sqrt{(1 - s^2)(k^2 - s^2)}} = \frac{1}{\sqrt{(1 - s^2)(k^2 - s^2)}} + \frac{2}{1 - k^2}\sqrt{\frac{1 - s^2}{k^2 - s^2}}$$

$$- 2\,\frac{d}{ds}\left[\frac{\sqrt{(1 - s^2)(k^2 - s^2)}}{(1 - k^2)(1 - s)}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (8c)$$

This step can be verified by differentiating the right-hand term of Eq. 8*b* as indicated, reducing all terms to a common denominator and combining. Fourth step: From Eqs. 8*b* and 8*c*,

$$z = M\left[\int \frac{ds}{\sqrt{(1 - s^2)(k^2 - s^2)}} + \frac{2}{1 - k^2}\int \sqrt{\frac{1 - s^2}{k^2 - s^2}}\,ds - 2\,\frac{\sqrt{(1 - s^2)(k^2 - s^2)}}{(1 - k^2)(1 - s)}\right] \qquad \text{(8d)}$$

Fifth step: In the two integrals on the right in Eq. 8*d*, make $s = k \sin \phi$. Then

$$z = M\left[\int \frac{d\phi}{\sqrt{1 - k^2 \sin^2 \phi}} + \frac{2}{1 - k^2}\int \sqrt{1 - k^2 \sin^2 \phi}\,d\phi - \frac{2\sqrt{(1 - s^2)(k^2 - s^2)}}{(1 - k^2)(1 - s)}\right] \qquad \text{(8e)}$$

The two integrals are now the standard elliptic integrals of the first and second kind, respectively. Hence

$$z = M\left[F(\phi,k) + \frac{2}{1 - k^2}E(\phi,k) - \frac{2\sqrt{(1 - s^2)(k^2 - s^2)}}{(1 - k^2)(1 - s)}\right] + C_2 \qquad \text{(9)}$$

in which C_2 is the constant of integration.

To compute values of y and z the best method is to use ϕ as a parameter. Then, since $s = k \sin \phi$, Eq. 9 becomes

$$y = \frac{\sqrt{b^2 d^2 - 16\,C}}{2\,b} \sin \phi + \frac{d}{2} \qquad \text{(10)}$$

Then y and z are calculated[10] from Eqs. 9 and 10 respectively, using tables of $F(\phi,k)$ and $E(\phi,k)$. The streamlines have been plotted in Fig. 9, showing three views—section, plan, and elevation. Here the value $k_v = k_w = 1$ was used, merely to determine the curve forms, since any other relationship between k_v and k_w would only change the distance that a particle would have to move downstream to complete one cycle of the curves, but would not change their general shape.

Angular Velocity.—According to the laws of hydrodynamics the components of the angular velocity, ξ, η and ζ, about the x-axis, y-axis, and z-axis are, assuming time does not affect the velocities at a given section:

$$\xi = \frac{1}{2}\left(\frac{\partial w}{\partial y} - \frac{\partial v}{\partial z}\right) = (d - y)\left(\frac{b^2}{4} - x^2\right) \qquad \text{(11a)}$$

$$\eta = \frac{1}{2}\left(\frac{\partial u}{\partial z} - \frac{\partial w}{\partial x}\right) = x\,(2\,y\,d - y^2) \qquad \text{(11b)}$$

and

$$\zeta = \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right) = \frac{1}{2}(y\,d - y^2) + \left(\frac{b^2}{4} - x^2\right) \qquad \text{(11c)}$$

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[10] "Formules et Tables Numeriques aux Fonctions Circulaires, Hyperboliques, Elliptiques," by L. Potin, 1925, pp. 753–827.

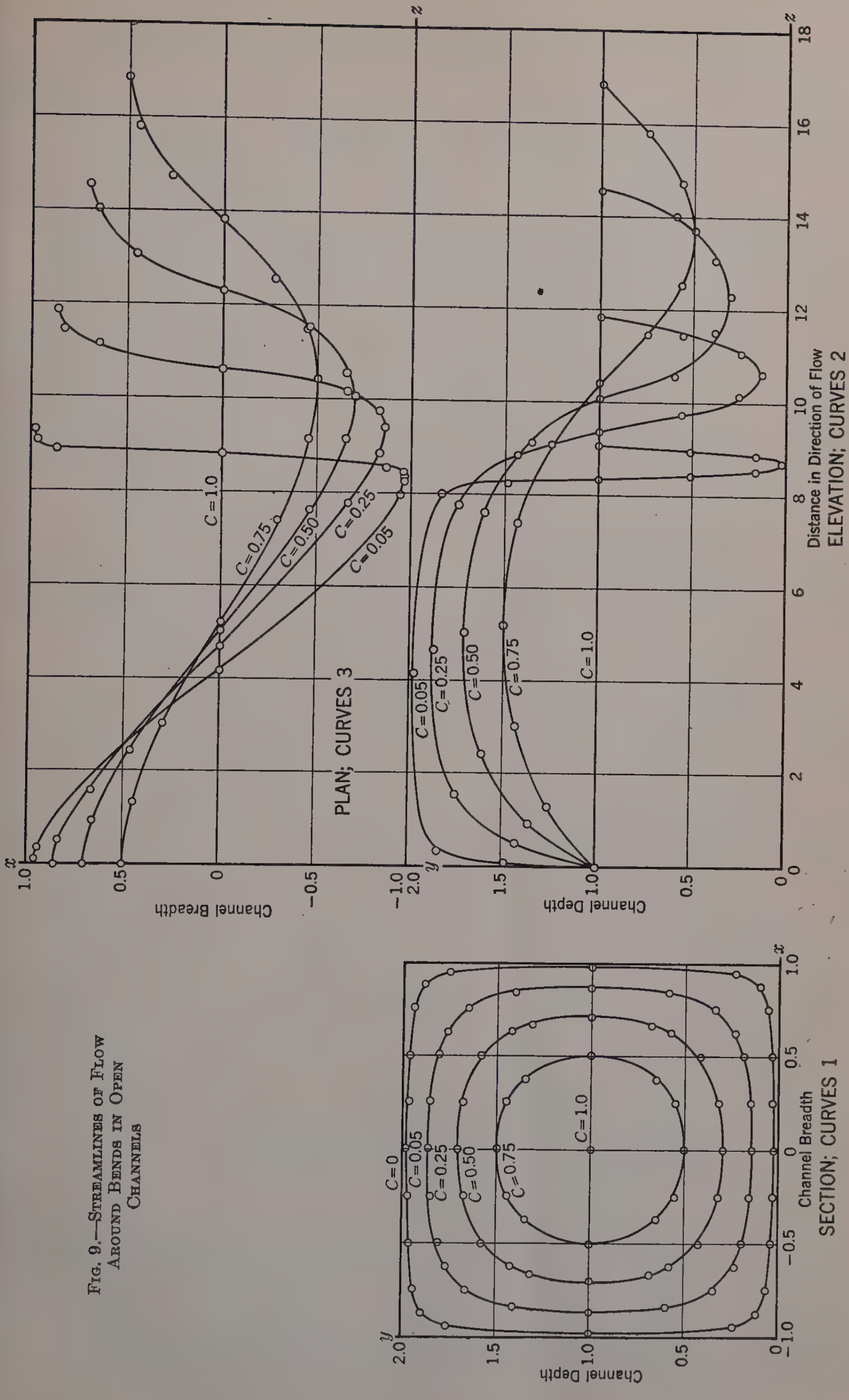


Fig. 9.—Streamlines of Flow Around Bends in Open Channels

Then the angular velocity in radians per second is:

$$\omega = \sqrt{\xi^2 + \eta^2 + \zeta^2}$$

$$= \sqrt{\left[(d-y)\left(\frac{b^2}{4}-x^2\right)\right]^2 + [x\,(2\,y\,d-y^2)]^2 + \left[\frac{1}{2}(y\,d-y^2)+\left(\frac{b^2}{4}-x^2\right)\right]^2} \quad ..(12)$$

A check on the accuracy of Eq. 12 may be had by noting that the divergence of the angular velocity must be zero: That is, a divergence of

$$\omega = \frac{\partial \xi}{\partial x} + \frac{\partial \eta}{\partial y} + \frac{\partial \zeta}{\partial z} = 0 = \frac{\partial}{\partial x}\left[(d-y)\left(\frac{b^2}{4}-x^2\right)\right] + \frac{\partial}{\partial y}[x\,(2\,y\,d-y^2)],$$

or

$$\omega = -\,2\,x\,(d-y) + 2\,x\,(d-y) = 0 \ldots\ldots\ldots\ldots\ldots (13)$$

In Eq. 12 it should be noted that the η-component is not affected by the breadth of the channel.

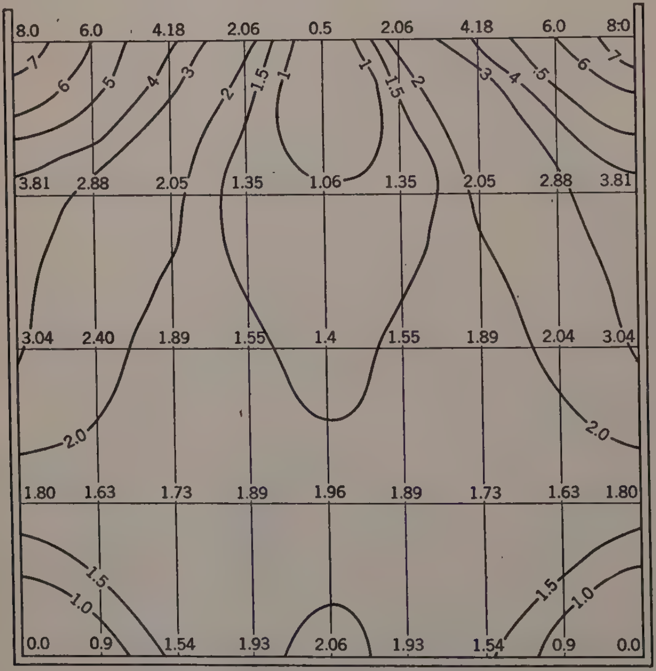
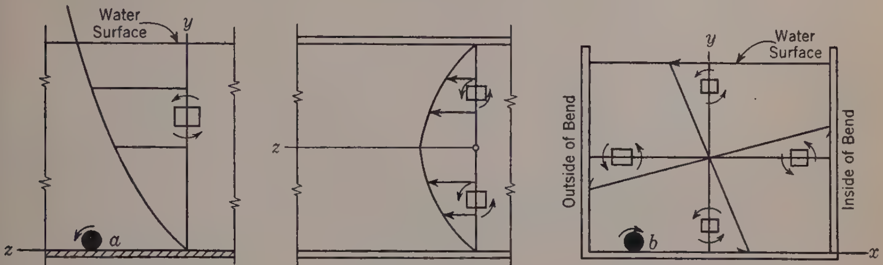


FIG. 10.—LINES OF EQUAL ANGULAR VELOCITY

Fig. 10 shows lines of equal angular velocity in the channel section, assuming $b = d = 2$, and $k_u = k_v = k_w = 1$. It is worthy of note that at values of $y = \frac{d}{4}$, the vorticity is fairly uniform all across the channel. The symmetry of the diagram is due to the original assumptions as to velocity components.

The direction of rotation of the components of the angular velocity can be seen by reference to Fig. 11. The direction of rotation in the y–z and y–x planes is always the same regardless of what point in the section is selected, whereas rotation in the z–x plane changes direction as a particle moves from the inside half of the bend through the center of the channel to the outside half of the bend.



(*a*) SIDE ELEVATION y - z PLANE (*b*) PLAN VIEW z - x PLANE (*c*) SECTION OF CHANNEL FACING UPSTREAM, y - x PLANE

FIG. 11.—DIRECTION OF ROTATION OF ANGULAR VELOCITY COMPONENTS

Acceleration.—Components of the acceleration parallel to the x-axis, y-axis, and z-axis are given by the equations:

$$a_x = u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z} = -2x\left(\frac{d}{2}-y\right)^2\left(\frac{b^2}{4}-x^2\right) - x\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2) \ldots\ldots \tag{14a}$$

$$a_y = u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z} = \left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2) + x^2(y\,d-y^2)(d-2y) \ldots\ldots \tag{14b}$$

and

$$a_z = u\frac{\partial w}{\partial x} + v\frac{\partial w}{\partial y} + w\frac{\partial w}{\partial z} = -2x(2y\,d-y^2)\left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right) + 2xy(d-y)^2\left(\frac{b^2}{4}-x^2\right) \ldots\ldots \tag{14c}$$

$$A = \sqrt{a^2_x + a^2_y + a^2_z} = \left\{\left[-2x\left(\frac{d}{2}-y\right)^2\left(\frac{b^2}{4}-x^2\right) - x\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2)\right]^2 + \left[\left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2) + x^2(y\,d-y^2)(d-2y)\right]^2 + \left[-2x(2y\,d-y^2)\left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right) + 2xy(d-y)^2\left(\frac{b^2}{4}-x^2\right)\right]^2\right\}^{0.5} \ldots\ldots \tag{15}$$

Table 2 shows components of the acceleration parallel to the x-axis, y-axis, and z-axis for different values of x, y, and z in terms of b and d, assuming

$k_u = k_v = k_w = 1$. It should be noted that, at all points in the section, a_x varies as $b^3\,d^2$; a_y varies as $b^2\,d^3$; and a_z varies as $b^3\,d^3$. This is significant where, as in most natural streams, breadth is greater than depth, and hence the

TABLE 2.—COMPONENTS OF ACCELERATION, a,

Values of x(a)	$y = 1.00\,d$			$y = 0.75\,d$			
	a_x	a_y	a_z	a_x	a_y	a_z	a_x
$0.50\,b$	0	0	0	0	$-3\,b^2\,d^3/128$	0	0
$0.25\,b$	$3\,b^3\,d^2/128$(b)	0	$-3\,b^3\,d^3/64$(b)	$-3\,b^3\,d^2/1{,}024$(b)	$-33\,b^2\,d^3/1{,}024$	$-24\,b^3\,d^3/1{,}024$(b)	$3\,b^3\,d^2/256$(b)
0(c)	0	0	0	0	$-3\,b^2\,d^3/256$	0	0

(a) Values of x in terms of channel width, b. (b) All values in this table are for the outside half of the indicated by the footnote index. (c) Center line of channel.

acceleration across the channel and parallel to the x-axis is much greater than that in the vertical direction parallel to the y-axis. Likewise, the maximum u-component of the velocity, parallel to the x-axis, would exceed the v-component, parallel to the y-axis, and be in direct ratio of width to depth of stream.

On the outside half of the channel the velocity components parallel to the x-axis, and in the lower half of the section, are directed toward the inside of the bend and are increasing in magnitude as the particle moves toward the center of the channel. If a piece of gravel or sand should start moving along the bed of the stream in this vicinity, it would tend to continue and, after passing the middle of the stream, would tend to be stopped due to the decelerating components of the filamental velocities directed toward the inside of the bend.

The components of the angular velocity are helpful in understanding the movement of detritus along the bed of the stream. If, in Eq. 12, y is taken as zero, the angular velocity becomes:

$$\omega = \sqrt{\left[d\left(\frac{b^2}{4} - x^2\right)\right]^2 + 0 + \left[\frac{1}{2}\left(\frac{b^2}{4} - x^2\right)\right]^2} \quad \ldots\ldots\ldots (16)$$

Hence, under the assumptions made, there is no tendency for the gravel on the stream bed to rotate about a vertical axis, but there are components of angular velocity about the x-axis and z-axis. The rotation about the x-axis is the same as the direction a piece of gravel would be obliged to roll along the stream bed, and although the rotation about the z-axis, parallel to the axis of the center line of the channel (Figs. 11(a) and 11(b)), is retrograde to the direction a piece of gravel would be obliged to rotate, its magnitude varies as $\frac{b^2}{8}$ compared to $\frac{b^2\,d}{4}$ for the value of $x = 0$ at the center of the stream. When $b = d = 2$ and $k_u = k_v = k_w = 1$, the one component is four times the other in value. However, in a natural channel it is likely that the maximum cross-currents are at some distance above the bed of the stream, in which case the angular velocity would not be retrograde to the direction that a piece of gravel would roll.

The coefficients accompanying the components of the acceleration in Table 2 are based on the assumption $k_u = k_v = k_w = 1$, which is not necessarily true, although it can be shown that $k_u = k_v$, since the divergence of the velocity $= 0 = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z}$; and that $\frac{\partial}{\partial x}\left[k_u\left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right)\right]$ $= -\frac{\partial}{\partial y}\left[k_v\, x\,(y\,d - y^2)\right]$; or $k_u = k_v$.

AT VARIOUS POINTS IN THE CHANNEL SECTION

$y = 0.50\,d$		$y = 0.25\,d$			$y = 0$			Values of x[(a)]
a_y	a_z	a_x	a_y	a_z	a_x	a_y	a_z	
0	0	0	$3\,b^2\,d^3/128$	0	0	0	0	$0.50\,b$
0	$-3\,b^3\,d^3/256$[(b)]	$15\,b^3\,d^2/1{,}024$[(b)]	$15\,b^2\,d^3/1{,}024$	$-3\,b^3\,d^3/1{,}024$[(b)]	$3\,b^3\,d^2/128$[(b)]	0	0	$0.25\,b$
0	0	0	$b^2\,d^3/64$	0	0	0	0	0[(c)]

channel. Values for the inside half are symmetrically similar except that the sign is changed in the cases

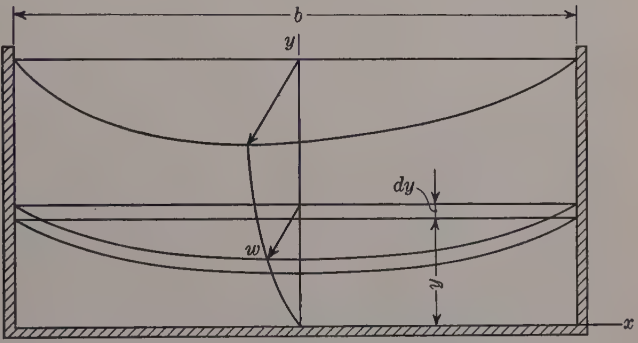


FIG. 12.—VELOCITY COMPONENT ALONG THE z-AXIS

The relation of these coefficients to k_w depends upon how far downstream a particle must move to complete a cycle of the streamlines shown in Fig. 9. However, some notion of the value of k_w may be had from the following analysis: From Fig. 12 it is apparent that when $x = 0$, $dQ = \frac{2}{3}\,b\,w\,dy$ $= \frac{2}{3}\,b\,k_w\left(\frac{b^2}{4} - x^2\right)(2\,y\,d - y^2)\,dy$. When $x = 0$,

$$dQ = \frac{k_w\,b^3}{6}\,(2\,y\,d - y^2)\,dy \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Integrating,

$$Q = \frac{k_w\,b^3}{6}\int_0^d (2\,y\,d - y^2)\,dy = \frac{k_w\,b^3\,d^3}{9} \ldots\ldots\ldots\ldots (18)$$

By use of Eq. 18 it is possible to evaluate k_w if the rate of flow Q, breadth of channel b, and depth of channel d are known.

At this point it is of interest to note the ratio of the maximum to the average velocity, w, in the direction of the z-axis. The maximum w-component is: When $x = 0$, and $y = d$,

$$w\ (\text{max}) = k_w \left(\frac{b^2}{4} - x^2\right)(2\,y\,d - y^2) = \frac{k_w\,b^2\,d^2}{4} \quad \ldots\ldots\ldots (19a)$$

$$w\ (\text{ave}) = \frac{Q}{b\,d} = \frac{k_w\,b^2\,d^2}{9} \quad \ldots\ldots\ldots (19b)$$

and

$\dfrac{w\ (\text{max})}{w\ (\text{ave})} = \dfrac{k_w\,9\,b^2\,d^2}{k_w\,4\,b^2\,d^2} = \dfrac{9}{4} = 2.25.$ This would indicate that the maximum value of the velocity along the axis of the stream is 2.25 times as large as the average, on the basis of the assumptions made in this paper.

FREE VORTEX FLOW

Now suppose the w-component of the velocity complies more nearly with the law of a free vortex as shown in Fig. 13. Referring to Figs. 13(b) and 13(c), $k_7\,d^2 - k_7\,(d - y)^2 = k_7\,(2\,y\,d - y^2)$ and $w_i\,r_i = w_o\,r_o =$ a constant.

At the water surface

$$w = \frac{k_8}{r} \quad \ldots\ldots\ldots (20a)$$

and at any point—

$$w = \frac{k_7\,k_8\,(2\,y\,d - y^2)}{r_c - x} = \frac{k_w\,(2\,y\,d - y^2)}{r_c - x} = \frac{k_w\,(2\,y\,d - y^2)}{1 + \frac{b}{2} - x} \quad \ldots\ldots (20b)$$

If u and v remain the same as before, and $k_u = k_v = k_w = 1$, the components of the filamental velocities parallel to the x-axis, y-axis, and z-axis will be:

$$u = \left(\frac{d}{2} - y\right)\left(\frac{b^2}{4} - x^2\right) \quad \ldots\ldots\ldots (21a)$$

$$v = x\,(y\,d - y^2) \quad \ldots\ldots\ldots (21b)$$

$$w = \frac{2\,y\,d - y^2}{1 + \frac{b}{2} - x} \quad \ldots\ldots\ldots (21c)$$

Streamlines.—In the x–y plane the streamlines are the same as those computed under the first assumptions. The general equation is obtained as follows:

$$\frac{dz}{2\,y\,d - y^2} \div \left(1 + \frac{b}{2} - x\right) = \frac{dy}{x\,(y\,d - y^2)} \quad \ldots\ldots\ldots (22a)$$

and

$$dz = \frac{(2\,d - y)\,dy}{x\,(d - y)\left(1 + \frac{b}{2} - x\right)} \quad \ldots\ldots\ldots (22b)$$

Substituting from Eq. 5, $x = \left(\frac{b^2}{4} - \frac{C}{y\,d - y^2}\right)^{0.5}$ and

$$z = \int \frac{(2\,d - y)\,dy}{\left(\frac{b^2}{4} - \frac{C}{y\,d - y^2}\right)^{0.5} (d - y)\left[1 + \frac{b}{2} - \left(\frac{b^2}{4} - \frac{C}{y\,d - y^2}\right)^{0.5}\right]} + C_3 \text{..(23)}$$

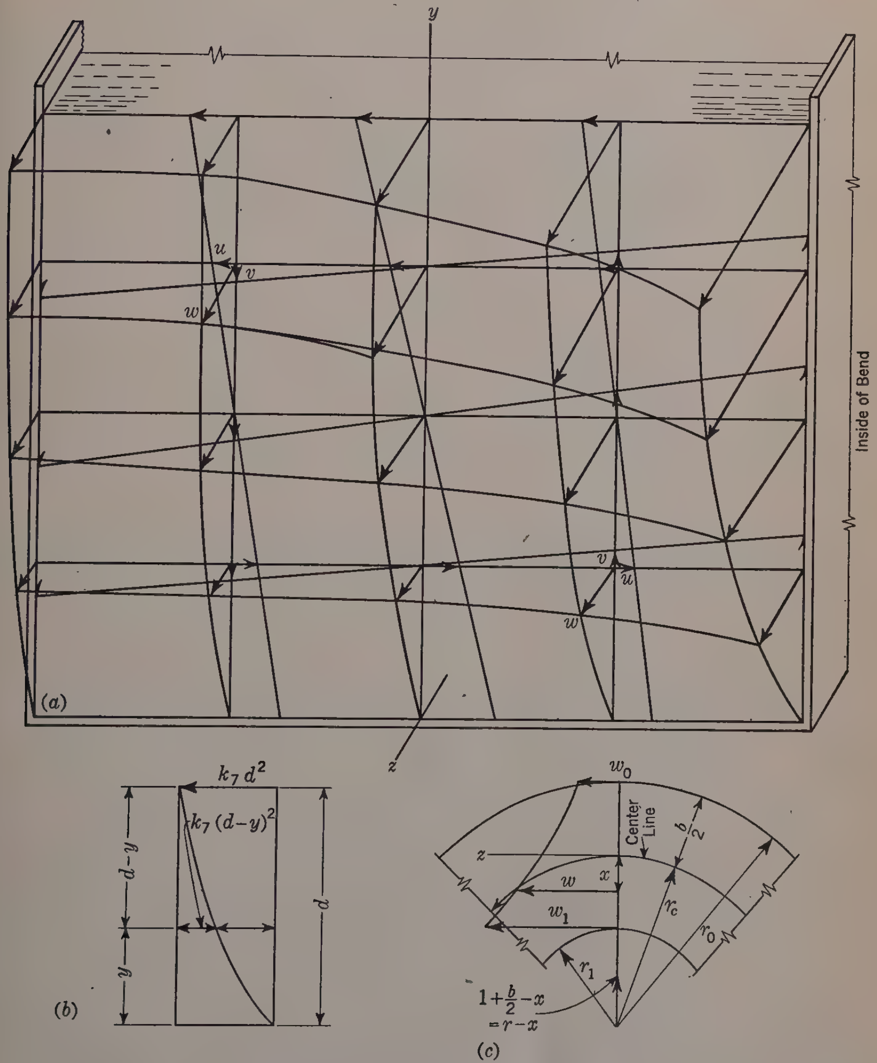


Fig. 13.—Flow Conditions for Free Vortex Flow

It appears difficult to evaluate this integral, and perhaps it is not necessary since some notion of the nature of the streamlines may be had by reference to those for the first assumptions.

Angular Velocity.—For the revised assumptions as to velocity distribution, the general equation for angular velocity becomes:

$$\omega = \sqrt{\left[\frac{d-y}{1+\frac{b}{2}-x}\right]^2 + \left\{-\frac{y}{2}\left[\frac{2\,d-y}{\left(1+\frac{b}{2}-x\right)^2}\right]\right\}^2 + \left[\frac{y}{2}(d-y)+\frac{1}{2}\left(\frac{b^2}{4}-x^2\right)\right]^2} \quad \ldots (24)$$

The divergence of this angular velocity was checked and found to be zero, as it should be. The component in the y–x plane is the same as under the first assumptions, and, although the component in the y–z plane is changed, its direction has not been altered. The angular velocity in the z–x plane has the same direction at all points in the section; that is, it will not change sign from one side of the channel to the other as under the first assumptions.

Acceleration.—For the free vortex flow the acceleration becomes:

$$\begin{aligned} a = \Bigg\{ & \left[-2\,x\left(\frac{d}{2}-y\right)^2\left(\frac{b^2}{4}-x^2\right) - x\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2)\right]^2 \\ & + \left[\left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right)(y\,d-y^2) + x^2\,(y\,d-y^2)\,(d-2\,y)\right]^2 \\ & + \left[\frac{\left(\frac{d}{2}-y\right)\left(\frac{b^2}{4}-x^2\right)(2\,y\,d-y^2)}{\left(1+\frac{b}{2}-x\right)^2} + \frac{2\,x\,(y\,d-y^2)\,(d-y)}{1+\frac{b}{2}-x}\right]^2\Bigg\}^{0.5} \quad . \quad (25) \end{aligned}$$

The components of the acceleration parallel the x-axis and y-axis, and, as given by Eq. 25, have not been changed over what they were under the original assumptions; hence the projection of the streamlines on the z–y plane is the same as that shown in Fig. 9. The z-component of the acceleration, parallel to the axis of the stream, does not now have as large values as before. If time and care were taken to compare velocities and accelerations as indicated by equations here developed, a suitable comparison between the streamlines of Fig. 9 and those indicated by Eq. 23 might be made.

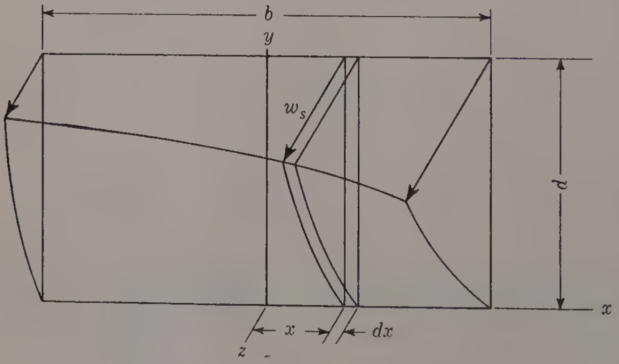


FIG. 14

It is possible to analyze the relation between the maximum and the average w-component of the velocity, similar to that under the original assumptions. If w_s is taken as the w-component of the velocity at the water surface, then (see Fig. 14):

$$dQ = \frac{2}{3}\, d\, w_s\, dx = \frac{2}{3}\, d\, k_w \left(\frac{2\,y\,d - y^2}{r_c - x}\right) dx \qquad (26a)$$

When $y = d$

$$dQ = \frac{2\,k_w\,d^3}{3\,(r_c - x)}\, dx \qquad (26b)$$

and

$$Q = \frac{2}{3}\, k_w\, d^3 \int_{-0.5\,b}^{0.5\,b} \frac{dx}{r_c - x} = -\frac{2}{3}\, k_w\, d^3 \log_e \left(\frac{2\,r_c - b}{2\,r_c + b}\right) \qquad (27)$$

The maximum value of the w-component of the velocity occurs when $y = d$ and $x = \frac{b}{2}$; then

$$w = \frac{k_w\,(2\,y\,d - y^2)}{r_c - x} \qquad (28a)$$

$$w\,(\max) = \frac{k_w\,d^2}{r_c - 0.5\,b} = \frac{2\,k_w\,d^2}{2\,r_c - b} \qquad (28b)$$

and

$$w\,(\text{ave}) = \frac{Q}{b\,d} = -\log_e \frac{2\,r_c - b}{2\,r_c + b}\left(\frac{2\,k_w\,d^2}{3\,b}\right) \qquad (28c)$$

Finally,

$$\frac{w\,(\max)}{w\,(\text{ave})} = \frac{-3\,b}{(2\,r_c - b)\log_e \dfrac{2\,r_c - b}{2\,r_c + b}} \qquad (29)$$

In the laboratory experiment where $r_c = 1.75$ ft and $b = 1.5$ ft, the ratio indicated by Eq. 29 has the value of 2.457, and is about 10% higher than under the first set of assumptions as to filamental velocities. It is of interest to note that Eq. 29 is not affected by the depth of the channel, the only variables in the right-hand member being breadth of channel and radius of curvature.

Eq. 27 can be used to evaluate k_w when the rate of flow and the breadth, depth, and radius of curvature of channel are known. For the laboratory experiment when the rate of flow $Q = 0.449$ cu ft per sec, $b = 1.50$ ft, $d = 0.467$ ft, and $r_c = 1.75$ ft, k_w has the value of 7.22. Using this value of k_w, computations were made for the theoretical velocities at 0.6 d and plotted against values scaled from velocity contours obtained from laboratory data as shown in Fig. 2(*a*). The laboratory data were for a section of the channel taken one-half way around a 180° bend.

Some justification may be had for assuming the w-component of the filamental velocities to vary according to the law of a free vortex. (Fig. 15 shows values of the velocity scaled from Fig. 16, Section 12, Stage 0.496, at 0.6 depth, and $k_w = 7.2$.) The computed values of the velocities are higher than the actual values near the water surface, and are smaller near the bottom of the channel. This difference perhaps could have been made less by taking the origin of the parabola, Fig. 15, Section A–A, at the bottom of the channel instead of at the water surface as was done in the original assumptions; but there seems to be no really typical vertical velocity curve for all open channels,[11] and if one were to be selected for a rectangular channel in actual service it

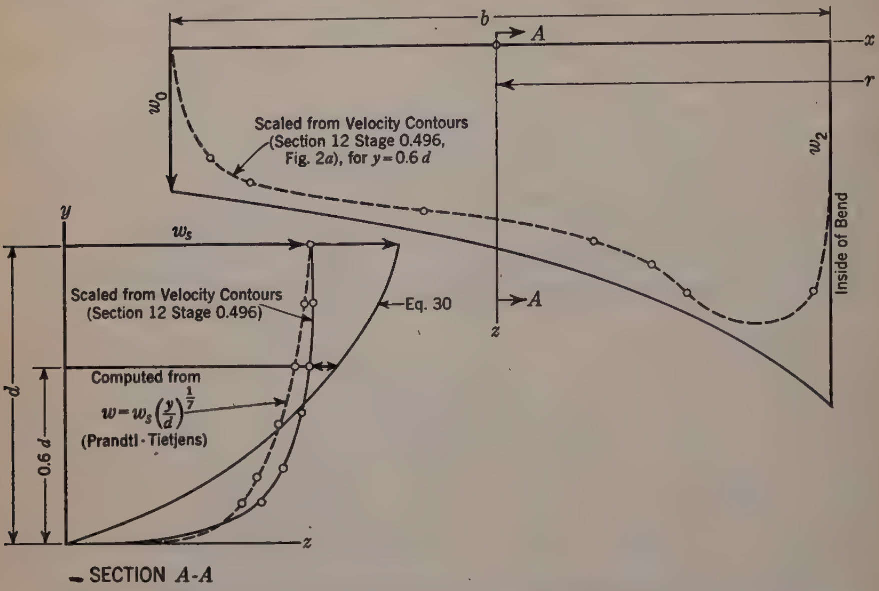


FIG. 15.—COMPARISON OF ACTUAL TO COMPUTED VALUES OF w-COMPONENTS OF VELOCITIES

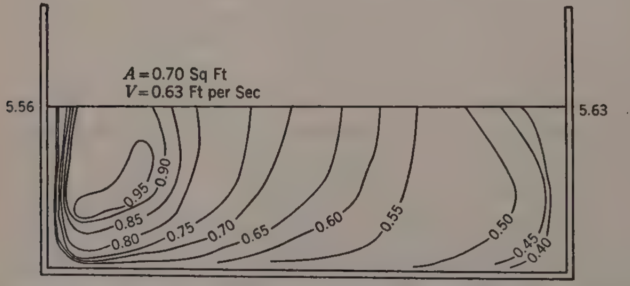


FIG. 16.—STAGE 0.496, SECTION NO. 12, RATE OF FLOW CONSTANT (0.449 CU FT PER SEC)

would not be far different from the one here assumed, judging from the actual curves reported by F. C. Scobey,[12] M. Am. Soc. C. E., for flow in irrigation canals.

In Fig. 15 is also shown a curve proposed by O. Tietjens,[13] in accordance with the equation

$$w = w_s \left(\frac{y}{d}\right)^{1/7} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

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[11] *Water-Supply Paper No. 868-A*, U. S. Geological Survey, Plate 4.

[12] "The Flow of Water in Irrigation Canals," by Fred C. Scobey, *Bulletin No. 194*, U. S. D. A., 1915, p. 18.

[13] "Hydro- und Aeromechanik," by L. Prandtl and O. Tietjens, *Zweiter Band*, Springer, Berlin, 1931, p. 87.

Although this compares favorably with the actual velocity measurements, it does not exactly fit, and to make use of such a law for the mathematical definition of the velocity components would lead to an equation of the streamlines that would be most difficult if not impossible to evaluate.

The curve for Eq. 30, for $x = 0$, $r_c = 1.75$ ft, $d = 0.467$ ft, $b = 1.50$ ft, and $k_w = 7.2$, is also included in Fig. 15.

CONCLUSIONS

This research was undertaken as a study of the flow conditions in the bend of an open channel. Many observations have indicated that the spiral exists in the bends of both open and closed channels, and that in the latter the spiral is usually double. An attempt has been made to observe the direction and magnitude of the angular velocities of particles in suspension as the water moved through the channel bend, and to compare these with theoretical analyses made under assumed flow conditions. The observed rotation of the suspended particles behaved in general with the theoretical analysis based on the assumed conditions. The path taken by a particle on its way around a bend in an open channel is extremely complicated and varied, but in general it must follow the pattern of a spiral.

In the experimental channel the crosscurrents moving along the stream bed from outside to inside of bend were amazingly strong compared to the average velocities in the stream. Evidence of this phenomenon was recorded in a motion picture for reference.

The following conclusions seem justified by the research:

1. Spiral flow in the bends of open channels as enunciated by James Thompson[2] certainly does exist, although the spiral pattern is an exceedingly complex one.
2. The downstream components of the filamental velocities in the first half of the bend are greater near the convex bank than at the concave bank and vary across the channel in close agreement with the law of a free vortex.
3. The angular velocities and accelerations inherent to the spiral motion constitute strong contributing factors to the movement of bed load, not only in a downstream direction but toward the inside of bends.
4. At about three-fourths the way around the bend there is a tendency for the development of an eddy, or slack water, along the inside bank due to the spiral motion, which is conducive to the deposition of suspended matter and the formation of a bar.

ACKNOWLEDGMENTS

Special acknowledgment is due W. E. Milne, head of the department of mathematics at Oregon State College, Corvallis, Ore., for the solution of the streamline equation, and to Alton E. Alspaugh and William B. Matlock for plotting the curves for the streamlines. The writer also wishes to give credit to Frank E. Lucas, Jun. Am. Soc. C. E., for help in the construction of the laboratory apparatus, and to all other individuals whose advice and counsel were so kindly given.

The funds for the purchase of expendable laboratory apparatus and miscellaneous materials for the project were furnished by the Engineering Foundation through J. C. Stevens, M. Am. Soc. C. E., chairman of the committee of the Society's Hydraulic Division on Hydraulic Research.

APPENDIX

NOTATION

The following letter symbols, used in this paper, conform essentially to American Standard Letter Symbols for Hydraulics, prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1942:[14]

a = acceleration for free vortex flow;
b = breadth of water stream;
C = constants of integration, C_1, C_2, etc.;
d = depth of stream;
F = a centrifugal force;
k = modulus of the elliptic integral: k_u, k_v, and k_w = velocity coefficients, referring to the u-component, v-component, and w-component of velocity;
L = length;
m = mass of water;
Q = rate of flow;
r = radius of curvature: r_c = radius of curvature at center of stream; r_i = inside radius; and r_o = outside radius;
T = time;
u = component of velocity, parallel to the x-axis;
V = average velocity across one given section;
v = component of velocity, parallel to the y-axis;
w = component of velocity, parallel to the z-axis;
x = lateral distance, measured parallel to the x-axis, across the channel;
y = vertical distance, measured parallel to the y-axis;
z = horizontal distance, measured parallel to the z-axis, along axis of channel;
ζ = angular velocity about the x-axis;
η = angular velocity about the y-axis;
ξ = angular velocity about the z-axis;
ϕ = amplitude of the elliptic integral;
ω = angular velocity.

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[14] ASA—Z10.2—1942.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

THE QUEENS MIDTOWN TUNNEL

By Ole Singstad,[1] M. Am. Soc. C. E.

Synopsis

The engineering features of the Queens Midtown Tunnel, including its function from the standpoint of automotive transportation, its planning, design and construction, are discussed. An attempt has been made, within the permissible limits of space, to emphasize certain features of the project which presented more than the usual problems.

Introduction

The Queens Midtown Tunnel is a subaqueous highway crossing of the East River in New York, N. Y. It extends in an east-west direction between the east side of the midtown section of the Borough of Manhattan and Long Island City in the Borough of Queens. Its location, highway connections, and relation to the New York City metropolitan area are shown in Fig. 1, which is

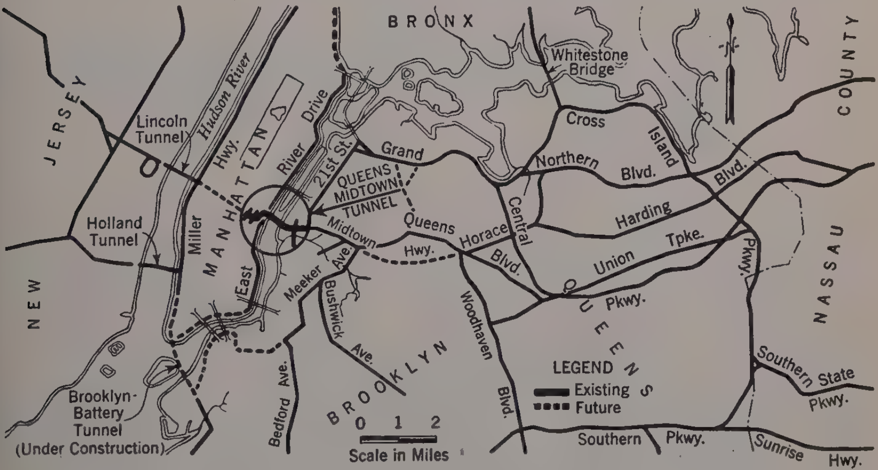


Fig. 1.—Highways and Parkways Tributary to the Tunnel as of 1943

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by August 1, 1943.

[1] Chf. Engr., New York City Tunnel Authority, New York, N. Y.

based on the present (1943) status of development of highways and parkways. The tunnel exits and entrances in Manhattan connect with the local street system and are conveniently situated with respect to the East River Drive, which extends along the east shore of Manhattan Island. In Queens, the principal connection to the tunnel is the Midtown Highway which, via the Connecting Highway, provides an express route to Queens Boulevard, the main central traffic artery of Queens. The latter, in turn, leads to and from the express parkway and highway system of Long Island. The Queens plaza also provides connections via 21st Street (Van Alst Avenue), 50th Avenue, and Borden Avenue to the local street system and areas in Brooklyn and Queens directly tributary to the tunnel.

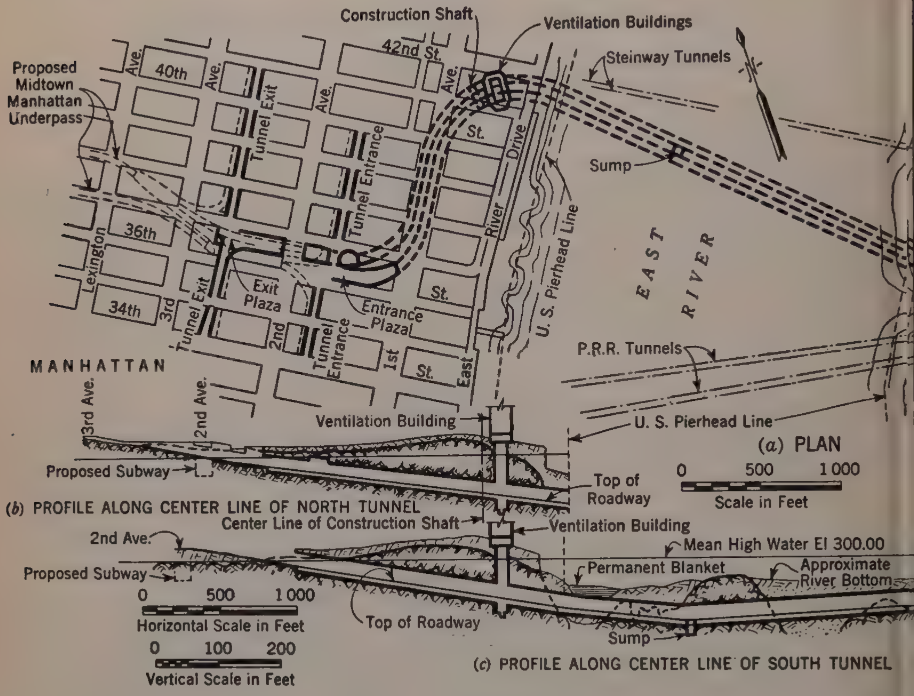
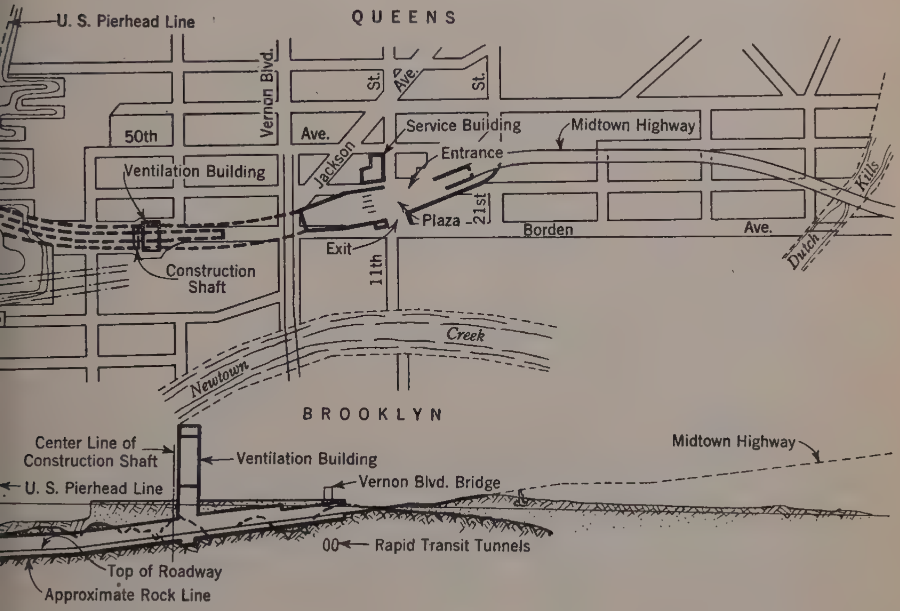


FIG. 2.—GENER

Fig. 2 shows the general plan and profile of the tunnel. Between portals, the tunnel structure includes two cast-iron-lined shield-driven tubes, 31 ft in diameter, 3,760 ft long, two structural steel-lined tubes in rock tunnel, 32 ft 10 in. in diameter, 1,051 ft long, and steel bent and concrete structure for a length of 387 ft in Manhattan and 1,051 ft in Queens. In all essential respects, the design of the tubes, interior details, and provisions for ventilation and lighting are of the same design as for the Holland Tunnel and the other modern tunnels for automotive traffic that have been designed and constructed according to the Holland Tunnel pattern. (The Holland Tunnel was built by the New York State Bridge and Tunnel Commission and New Jersey Interstate Bridge and Tunnel Commission under the Hudson River between Canal Street and Broome Street, Manhattan, and 12th Street and 14th Street, Jersey City, N. J. It was opened to traffic November 13, 1927.) The tubes have an interior lining of concrete with walls and ceilings finished in tile. The roadways, which are paved with brick, are 21 ft wide between curbs, increased to a maximum of 23.5 ft on curves, providing for two lanes of westbound traffic in the north tube and two lanes of eastbound traffic in the south tube. The roadway headroom is 13.5 ft.



N AND PROFILE

The tunnel plazas, paved with brick in areas adjacent to the portals and with concrete beyond those areas, widen from the two lanes at the portals to six lanes for each direction of travel. Ventilation equipment is housed in two buildings, one on each side of the river. Remote control of ventilation and lighting, offices for the supervisory staff, quarters for maintenance crew and tunnel police and equipment are provided for in the service building in Queens. A garage for the emergency wrecking tractors is located at each plaza. Booths (ten single and one double) for the collection of tolls in both directions of travel are situated on the Queens plaza, approximately 432 ft from the tunnel portals.

General data describing the tunnel are as follows:

Traffic.—Estimated maximum capacity, in vehicles per year	16,000,000
Dimensions.—Length of crossing between plazas, in feet (from street grade to street grade)—	
Westbound	7,865
Eastbound	7,400
Maximum Grades Between Portals (%).—	
Up	3.5
Down	4.0
Maximum Grades at the Plazas (%).—	
Manhattan entrance (down)	4.8
Manhattan exit (up)	4.7
Queens (up and down)	4.5
Structural Depths Below Mean High Water, in Feet.—	
At the Manhattan pierhead line	
Roadway	77.0
Top of tube	55.2
At the low point	
Roadway	94.3
Bottom of tube	103.5
At the Queens pierhead line	
Roadway	74.0
Top of tube	52.2
Cost.—	
Construction contracts	$38,888,701.38
Miscellaneous construction items	23,107.81
Operating equipment	195,897.89
Real estate (including acquisition expenses)	6,836,790.81
Interest during construction	2,858,888.90
Engineering, administration, legal, overhead, and miscellaneous expenses	5,269,946.54
Actual total cost of project	$54,073,333.33
Estimated cost of project	$58,365,000.00
Finance (*U. S. Public Works Administration*).—	
Grant in aid	$11,235,000.00
Forty-year loan, at 4%	$47,130,000.00

HISTORICAL

A preliminary plan for a tunnel between Long Island and mid-Manhattan, extending across town to the west side, was first officially prepared by the president of the Borough of Manhattan in January, 1926. The idea of such a tunnel was endorsed early in the same year by the Regional Plan Association, Inc. (then the Committee on the Regional Plan of New York and Its Environs). In 1923 this committee had included in its scheme for main highway routes in the New York district[2] an east and west traffic axis from Long Island to New Jersey across midtown Manhattan. This plan contemplated the use of the

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[2] "Highway Traffic," *Regional Survey of New York and Its Environs*, Vol. III, p. 29.

Queensborough Bridge as the East River link in the route; but it was proposed that two new tunnels across the Hudson River be located between 59th Street and the Holland Tunnel, which was then being constructed.

From 1926 to 1935, the 38th Street-East River Tunnel (as it was first designated) was strongly advocated by many interested civic and business associations and groups as a means of:

(*a*) Relieving traffic congestion on existing East River bridges, particularly the Queensborough Bridge;

(*b*) Providing a necessary modern express link for vehicular traffic joining Manhattan Island with the rapidly growing Borough of Queens, the Borough of Brooklyn, and the highway system of Long Island; and

(*c*) Stabilizing the midtown Manhattan area as a center for business, manufacturing, shopping, amusements, hotels, and residence.

The Department of Plant and Structures of the City of New York made preliminary engineering studies and rendered a report on the project in June, 1929. In July of the same year, the Board of Transportation was assigned the work of preparing plans and specifications and starting construction with an initial appropriation of $2,000,000. The financial depression following the stock market crash of 1929 resulted in the rescinding of the greater part of this appropriation and the suspension of work on the project in 1931, at which time a plan for the East River crossing had been formulated.

The act of the New York State Legislature creating the New York City Tunnel Authority set up the East River crossing of the original project as separate from the east-west crossing of Manhattan Island; and in January, 1936, the Public Works Administration (PWA) allocated $58,365,000 for the construction of the East River crossing, or Queens Midtown Tunnel, with the understanding that the city would construct an express connection from the tunnel plaza in Queens to the main highway system of Long Island, to be completed by the time the tunnel was opened to traffic. The first work of the Tunnel Authority was a restudy of the tunnel plan and the development and adoption of the plan shown in Fig. 2. Ground was broken for the project on October 2, 1936, by President Franklin D. Roosevelt, and, after a construction period of slightly more than four years, regular operation of the tunnel was started on November 15, 1940.

Tunnel Highways, Connections and Traffic

In conformity with the promise made by the city at the time the tunnel project was financed, the scheme of connections and highways in Queens contemplated the completion, simultaneously with that of the tunnel, of the extension of Midtown Highway as far east as the junction of Horace Harding Boulevard, Queens Boulevard, and Woodhaven Boulevard. Although the construction of this extension of the Midtown Highway has been deferred temporarily, its necessity is apparent, as the existing alternate route via Connecting Highway and Queens Boulevard has insufficient capacity to provide for the normal growth of local, tunnel, and other interborough traffic using it, and is, in other respects, unsuitable and inadequate as part of the main east-west route of the tunnel.

In Manhattan, where the congestion which now exists on the crosstown streets of the midtown area west of Third Avenue and east of Eighth Avenue is a serious impediment to vehicular travel, the proposed Midtown Manhattan Underpass will provide an express route between the Manhattan plazas of the Queens Midtown Tunnel, the West Side Express Highway, and the Lincoln Tunnel. Its construction, with that of the highway improvements planned in Queens, is necessary to complete the express highway system joining Long Island, Manhattan, and New Jersey by a route through the center of the metropolitan area, of which the Queens Midtown Tunnel is a part.

The Queens Midtown Tunnel is the seventh East River crossing for vehicular traffic to be constructed and the first tunnel crossing of the East River for automotive vehicles. Vehicular traffic crossing the East River had increased twentyfold from 4,500,000 trips per annum in 1910 to 90,000,000 trips per annum in 1936. The Queensborough Bridge, which is the most heavily traveled of these crossings, increased in vehicular traffic from 769,000 trips in 1910 to a maximum of 36,000,000 in 1933, a toll-free volume approximately equal to that of all the Hudson River crossings from the Battery to Tarrytown. The Queens Midtown Tunnel, charging a base toll of 25¢ for passenger cars and light trucks, functions principally as an express alternate to relieve this load.

Maximum peak hour traffic recorded for the tunnel to date (March, 1943) occurred on Wednesday, December 24, 1941, between 8 a.m. and 9 a.m., when 2,075 cars used the westbound tube. This peak was easily handled with five of the toll booths on the Queens plaza, and smooth flow of traffic through the tunnel was maintained, giving a practical demonstration of the adequacy of entrance and exit plazas and approaches.

Tunnel Plan

The East River crossing, or the present plan of the Queens Midtown Tunnel, was authorized for construction in 1935, after careful consideration had been given to prior plans such as those of the president of the Borough of Manhattan (1926) and the Board of Transportation (1931). The Lincoln Tunnel under the Hudson River was already being built. Every one who had investigated these projects from the first inception had agreed that the East River and Hudson River crossings should be located so that an under-Manhattan connection could be conveniently provided. Therefore very little choice was possible in the location of the Queens Midtown Tunnel. The location was further limited by the Pennsylvania Railroad tunnels at 33d Street and by the Steinway Tunnel at 42d Street.

That this location was the correct one for both the Hudson and East River tunnels was established by traffic surveys made on both rivers showing that the midtown section was the one which was most urgently in need of traffic relief on both rivers; and, although the general location of the Queens Midtown Tunnel was established by legislative enactment, there was real justification for such location. The East River crossing received intensive study by the

Tunnel Authority, and a great many study layouts were made before the final plan was adopted. All of the earlier plans developed by other city departments showed approach plans for the tunnel on the Manhattan side which were deemed unsatisfactory from the point of view of the free and expeditious handling of traffic without congestion.

Conditions affecting the choice of plaza and approach locations in Manhattan included the character of the topography, real estate costs, and street and tunnel traffic considerations. These conditions limited practical locations of the Manhattan plazas to the area south of 40th Street between Third Avenue and the East River where the ground surface is comparatively flat and low in elevation, where values of real estate were generally within allowable limits, and where traffic on crosstown streets is not heavy. West of Third Avenue the street grades rise more rapidly into the Murray Hill section of Manhattan, increasing, in some instances, to a rate of almost 7%. The high value of real estate in certain blocks eliminated these from consideration as possible sites for the plazas, and, in general, the values of real estate increase, as the center of the midtown section is approached, to prohibitive sums in so far as the allowable costs of the tunnel project were concerned. Local traffic on the crosstown streets east of Third Avenue is comparatively light, whereas, if the tunnel plazas were built farther west, the great increase in volume of traffic would seriously affect the convenience of travel.

The War Department required certain minimum depths of channel and tunnel structure between U. S. pierhead lines, the prescribed minimum depth of the top of the tunnel tubes at the Manhattan pierhead lines being 48 ft below mean low water with a minimum depth of channel of 38 ft below mean low water. To provide a minimum cover of 13 ft over the tubes, the depth of the structure at the pierhead line was actually fixed at 51 ft below mean low water. The crown of the tunnel roadway is 21.77 ft below top of tube. It would have been impossible to reach street grade at the desired points with direct alinement from the depth of roadway so fixed at the pierhead line and stay within maximum permissible tunnel grades. Accordingly, a plan was adopted in which the line of the East River crossing from Borden Avenue, in Queens, extends to a point at the Manhattan pierhead line between the lines of 41st Street and 42d Street extended, the necessary distance being developed along First Avenue to the plaza locations between 36th Street and 37th Street. This crossing of the river is no greater in length between pierhead lines than the direct one.

The entrance and exit plazas in Manhattan are separated so as to avoid concentrating tunnel traffic in too small an area, and to avoid conflict between traffic bound for the tunnel and that from it. Wider separation was not practicable, as moving the exit plaza north would have resulted in steeper grades and greater real estate costs, whereas restriction on the use of park property prevented moving the entrance plaza farther south. The exit plaza passes under Second Avenue above the level of the proposed Second Avenue subway; it is bridged between First Avenue and Second Avenue to permit tunnel-bound traffic to reach the entrance plaza from points north of 37th Street.

The surface approaches are three lanes wide, those for exit extend between Second Avenue and Third Avenue connecting with all crosstown streets from 34th Street to 41st Street, inclusive, with the exception of 36th Street, where there is an underpass for southbound tunnel traffic to prevent conflict with traffic on 36th Street bound for the tunnel. The entrance approaches extend between First Avenue and Second Avenue and connect with all crosstown streets between 34th Street and 40th Street, inclusive. By this scheme of surface approaches connecting with the plazas, tunnel traffic is not concentrated on any one crosstown street, and does not conflict with the traffic on the heavily traveled north and south avenues.

The Manhattan plazas and approaches of the tunnel are so designed as to permit the construction, at any time in the future, of plazas and approaches for the Midtown Manhattan Underpass, with provision for direct connections between the underpass and the tunnel.

In Queens, real estate values were relatively low and the slope of the terrain was very gradual east from the river, reaching an elevation of 15 ft above mean high water at Borden Avenue and 21st Street.

Under these favorable conditions, the most advantageous location for the plaza was found north of Borden Avenue and east of Vernon Boulevard and Jackson Avenue (see Fig. 2), where connections to Midtown Highway and 21st Street (Van Alst Avenue) were conveniently made.

At the U. S. pierhead line on the Queens side, the War Department prescribed that, with due consideration for the thickness of permanent protective covering decided necessary, the depth of top of tubes below mean low water should be a minimum of 43 ft. Street level at the plaza was easily reached with suitable grades from a depth at the pierhead line 5 ft greater than the prescribed minimum. The lowering of the tunnel profile below the minimum by this amount provided additional thickness of rock cover in the full rock sections and permitted a greater length of tunnel to be excavated entirely in rock.

The results of studies for the location of ventilation shafts and buildings on either side of the East River placed the shaft and building in Queens in Borden Avenue about 50 ft east of Front Street and, in Manhattan, about 250 ft east of First Avenue between 41st Street and 42d Street. This original location of the Queens shaft contemplated the caisson method for its construction. When bids were received for this work, which was included in the contract for the river tunnels, it was found that a substantial saving could be effected by moving the shaft about 200 ft farther east where it could be constructed in the open-cut section, and, by agreement with the contractor, this change of location and design was made.

The low point of the tunnel is approximately 1,168 ft east of the Manhattan ventilation shaft in the rock reef near the middle of the East River, making it possible to construct the midriver sump and pump chamber entirely in rock between tubes.

On the line of the river crossing of the tunnels, the west channel of the East River reaches a depth of 66 ft below mean high water. This depth would have required lowering the tunnel profile considerably to maintain a suitable cover over the tunnel, thus increasing the air pressures necessary for construction. Furthermore, as this channel depth occurred near the Manhattan pierhead line, it would have increased the length of line necessary to reach the surface in Manhattan. The profile of the tubes at this point was made high enough to avoid these conditions, and for approximately 150 ft in each tube, the top is above the original bed of the river (maximum 8 ft). The War Department permit included provision for a permanent blanket of clay overlain with a protective layer of riprap of sufficient extent and depth to maintain a minimum cover of 13 ft over the tubes in this area.

Geologic Data

Test borings were made at the plaza and approach sites and along the line of the tunnels to determine the topography of the rock floor, the character and kind of rock and overburden, and, in the land areas, ground-water elevations. When rock was encountered, a core was taken of sufficient length to establish the material as ledge rock, generally a minimum of 10 ft.

Borings along the river crossing were staggered north and south of the tube locations and were generally not farther apart than 200 ft measured along the line of the tunnels. They were taken closer together where necessary, to develop the contour of the rock surface.

The rock floor was disclosed to be exceedingly irregular in both elevation and character, consisting of Manhattan schist under Manhattan and changing progressively to Inwood limestone, Fordham gneiss, Hell Gate dolomite, and Brooklyn injection gneiss along the tunnel line to Queens. There were zones of disintegrated rock lying at low points, generally at the contacts of one formation with another. The overburden was indicated to be mostly pervious glacial deposits consisting of sand, gravel, and numerous boulders. The face of tunnel excavation was shown to pass many times in and out of full rock, mixed ground, and soft ground.

Design of Structures

Lining for Shield-Driven River Tunnels.—Cast-iron lining of the usual design was chosen for the shield-driven tunnels as best adapted to resist shield thrusts and to provide rigidity and water tightness. The length of straight ring was 32 in., or 2 in. greater than the maximum length of ring heretofore used in the river tunnels around New York City. This longer ring was used with complete success and resulted in a saving of material and erection costs and in faster progress as compared with shorter ring lengths. A width greater than 32 in. was impracticable at the time because of foundry limitations; $1\frac{1}{4}$-in. and $2\frac{1}{2}$-in. taper rings, used to provide for vertical and horizontal curvature and corrections to alinement and grade, were a maximum of 32 in. in length.

Each ring was made up of fourteen segments and a key, and two weights of cast-iron lining were used. The heavier ring, as shown in Fig. 3, with 14-in. flanges and weighing approximately 22.7 tons per ring, was used in soft ground and mixed face or where the character or thickness of rock cover required it; the lighter ring with 11-in. flanges and weighing approximately 12.5 tons was used in rock face where the cover was adequate.

The segments making up the tunnel ring were bolted together with four high-strength steel bolts in each cross flange, and adjacent rings were bolted together with eighty-five high-strength steel bolts in the circumferential flange, the bolts for the heavy rings being $1\frac{3}{4}$ in. in diameter and those for the light rings, $1\frac{3}{8}$ in. in diameter. The specified yield point for the bolts was 85,000 lb per sq in., and the ultimate strength was 110,000 lb per sq in.

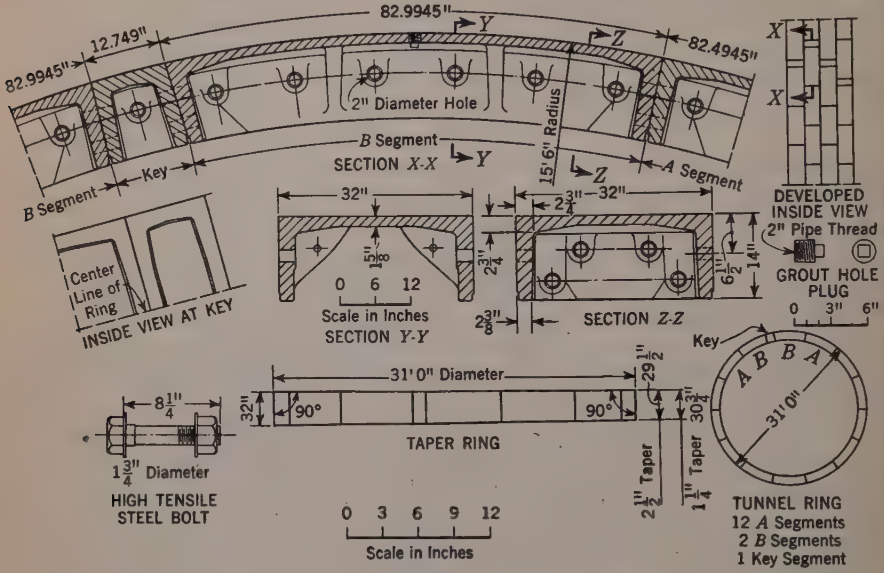


Fig. 3.—Heavy Cast-Iron Tunnel Lining Details

The load factors which were used in designing the heavy cast-iron lining are the weight of the cast-iron ring and that of the concrete lining; the full weight of earth and water above the horizontal diameter of the tunnel; lateral earth and water pressures; and the effect of buoyancy. The load factors used in designing the light cast-iron lining were the weight of the cast-iron and concrete linings, full hydrostatic pressure, and a load of 10 ft of rock packing above the top of the tunnel. The ring as thus designed was investigated for localized grout pressures.

The strength and efficiency of the segmental cast-iron ring depend on the capability of the bolts in the cross flanges to transmit the tunnel ring stresses between segments; and, with the cross joints of adjacent rings staggered, the bolts in the circumferential flanges connecting the adjacent rings, with proper bolt tightening, add materially to the strength of the cross joints by the splicing action provided.

An initial tension in the bolts of 25,000 lb per sq in. was specified. The bolts, which were erected with steel washers and hemp rope grommets, were tightened by hand, generally by two to three men on a ratchet wrench. A 5-ft wrench was used for the heavy cast-iron lining and a 4-ft wrench for the light lining. Actual bolt stresses, observed some time after the tunnel lining was erected, varied from intensities considerably less than 25,000 lb per sq in. to stresses approaching the yield point of the bolts. It is now considered good practice to make an allowance of about 5,000 lb per sq in. above the required bolt stress to offset any plastic yield effect in the entire bolt assembly under sustained loading.

Initial bolt stresses that are too low, and lack of uniformity in stress, may contribute to some leakage, especially in the case of the light cast-iron lining, as the flanges are thinner and more flexible than those of the heavy cast-iron lining and the surrounding material has no tendency to silt up any small crevices that may exist between flanges. Since even very small leaks may be objectionable as affecting the appearance of the tile-finished interior of a modern vehicular tunnel, it would be desirable to use machine tightening for the bolts in the light cast-iron lining. As a further safeguard against leakage, the light cast-iron lining may be designed with five bolts in the cross flanges. The fifth or center bolt of each cross flange has been omitted in recent tunnel design as theoretically carrying no stress.

The outside faces of all flanges of the Queens Midtown Tunnel lining were machined according to standard practice. The specified tolerance on the width of ring segments was very small (0 to $-\frac{1}{64}$ in.), in order to secure as tight a fit as possible along circumferential flanges. The lining was calked for watertightness with lead wire in the grooves formed at the inner edges of the flanges, the calking rebate on each segment being specified to have a maximum depth of $\frac{3}{16}$ in. so that a calking groove would be a maximum of $\frac{3}{8}$ in. in width when two segments were joined. Heavy machining on flange faces sometimes narrowed this groove so as to make calking difficult.

Permanent tunnel tie rods at ceiling and roadway levels were placed in the Queens Midtown Tunnel lining for short distances in both tubes under the Manhattan bulkhead area, where there might be changes in outside loadings after the tunnel was built which would possibly cause variations in the stresses in the lining. Permanent tunnel tie rods were also placed at the midriver sump, where certain segments of the tunnel lining were not erected because of the construction of the cross passage between tubes.

Midriver Sump and Pump Chamber.—The midriver sump and pump chamber are between the tunnels and consist essentially of two tubes, each one composed of segmented cast-iron rings with an interior lining of concrete. The axis of the pump chamber is at right angles to the main tunnels. The pump chamber rings are 16 ft in outside diameter, giving sufficient inside space, in addition to that required by the pumps, for a pedestrian cross passage between the main tunnels. The sump is 42 ft long, out to out, with rings of 14-ft outside diameter, and is located below the pump chamber with its axis parallel to, below and about midway between, the main tunnels. The pump chamber and cross passage are connected to the main tunnels by short rectangular structural steel frames and plates.

Rock Tunnels and Steel Bent and Concrete Structure, Manhattan.—In order to provide for the roadway widths which, because of the curved alinement, are increased to 23.5 ft in Manhattan, the metal lining, in the length of the rock tunnel sections extending from east of the Manhattan ventilation shaft to 38th Street, is designed with an outside diameter of 32 ft 10 in. These rock sections of tunnel were constructed as far as 41st Street under the contract for the river tunnels. Another contract was let for the structure from 41st Street to the tunnel portals, located just west of First Avenue between 36th Street and 37th Street, consisting of a continuation of the metal-lined rock tubes to 38th Street and steel bent and concrete structure from 38th Street to the portals. All of the metal lining, 32 ft 10 in. in diameter, was furnished under this contract, three alternative designs being submitted to the contractors for competitive bidding.

The three designs were based on the same strength factors, namely: To resist full hydrostatic pressure, plus a crown load of 10 ft of broken stone and grout estimated to amount to 1,500 lb per sq ft. One alternative was a lining 32 ft 10 in. in diameter, constructed of light cast-iron rings 32 in. long. The other two alternatives were linings of structural steel, designated as type 1 and type 2. Both type 1 and type 2 were rings of ten segments 5 ft in width and about 113 in. long and a key segment about 16 in. long. The weights of the two types were each about 2.2 tons per linear foot of tunnel. Type 1 was made of skin plates welded to circumferential and segmental flange plates 5¼ in. deep. The bulk of the stiffness and circumferential strength of the ring was provided by two split I-beams placed at the quarter points of the ring, bent to the tunnel radius, welded, and spliced with bolted connection plates at each cross joint to form a continuous circular rib. Type 2 also had segmental flange plates welded to the skin plate at each end, but the skin plate was bent to form circumferential flanges 9½ in. deep. One split I-beam ring was provided instead of two as on type 1.

Upon receipt of competitive bids, structural steel type 2 was found to be the least expensive of the three alternatives, and the contract was let on that basis. A third design in structural steel was submitted by the contractor (see Fig. 4). This design, which presented a saving in cost to the Tunnel Authority, differed from type 2 in that specially rolled bulb angles were provided for the circumferential flanges instead of bending up the skin plates. It was approved and installed.

The steel bent and concrete structure in Manhattan (Fig. 5(*a*)) extends from the circular tunnels, where the junction is made by bellmouths around the tubes, southwest to the plaza portals. The type of construction consists of structural steel bents spaced about 5 ft on centers down to the roadway level, below which the fresh air duct is of inverted reinforced concrete arch construction. Since the invert is in rock for its full length, this construction was found to be the most economical.

There is very little regularity to the steel bent and concrete structure in Manhattan. As the grade ascends to go into the plaza, the earth cover diminishes making it necessary to bring the exhaust air ducts from the top to one side of the structure. Special construction was necessary to form the junction between the circular tunnels and the rectangular section and also at the portals, where the structure is belled out to provide a gradual transition from tunnel lighting to daylight. Where the west tunnel passes under private property in the block between 38th Street and 37th Street, the structure was designed to support future 12-story storage warehouse buildings with floor live loads of 200 lb per sq ft.

Steel Bent and Concrete Structure, Queens.—The Queens land section was constructed entirely in trench from the construction shaft to the Queens plaza portal, a distance of 1,051 ft. For about one half of this length the structures for eastbound and westbound roadways were separated with a berm of earth and rock, and, for the remaining length, a single structure carries both roadways. The structural type is a steel bent spaced about 5 ft on centers, embedded in concrete and waterproofed with fabric and brick in asphalt mastic (see Fig. 5(*b*)).

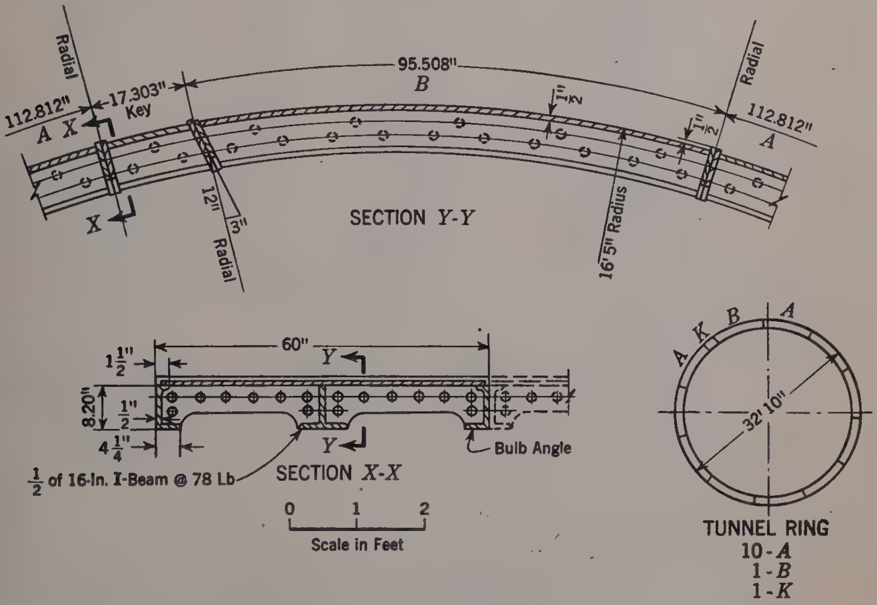
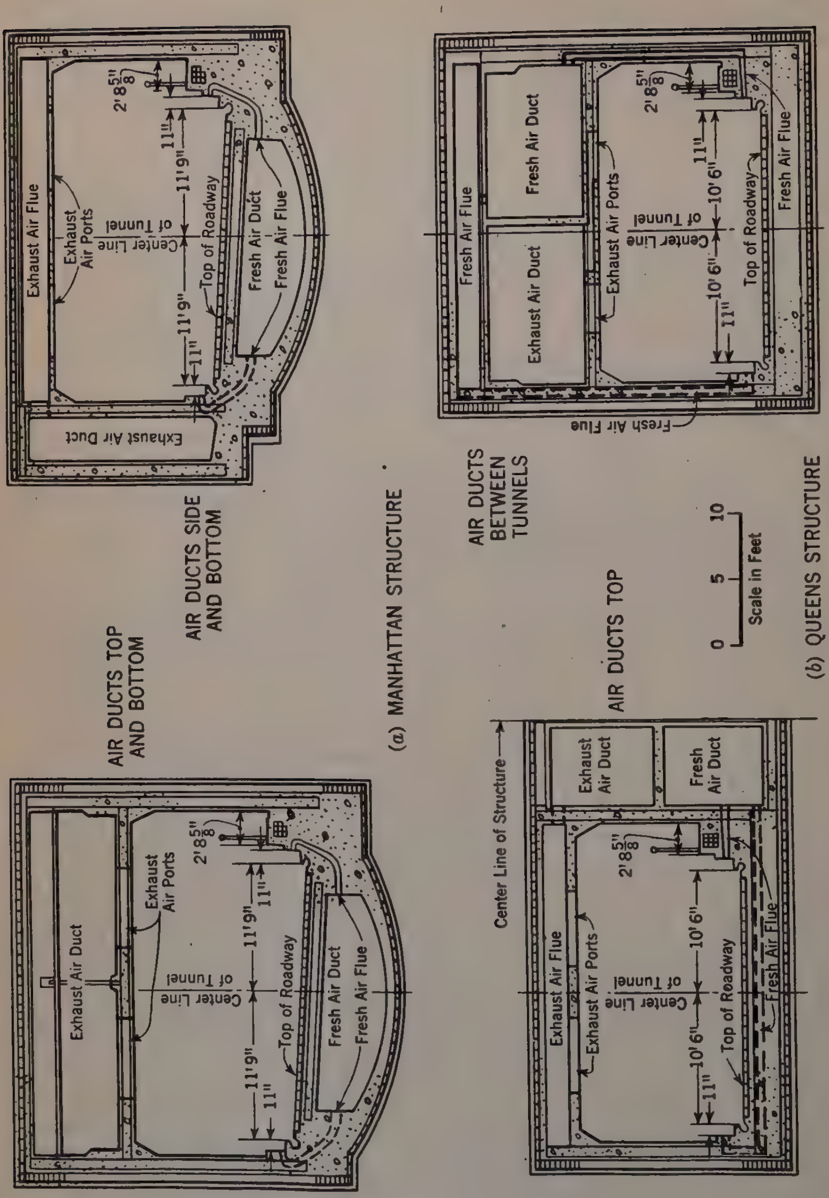


Fig. 4.—Steel Lining Details Designed for Use in Rock

The extreme west end of this land portion serves as the foundation for the Queens ventilation building, and provision was made in its design for all of the features normally located in a ventilation shaft—air ducts, emergency stairs, elevator, drainage sump, pump chamber, cable splicing chambers, etc.

This foundation structure was brought up to an elevation about 3 ft above the street surface and was designed to form the first floor of the ventilation building. Five pairs of steel girders, 5 ft in depth, distribute the building column loadings to the tunnel structure below and to ten circular steel-shell concrete-filled piers, varying from 4 ft to 4.5 ft in diameter, five located on each side of the tunnel structure and extended down to rock.

The depth of the structure made it possible to effect a substantial economy by placing the fresh air ducts above the ceiling for a large portion of the section instead of in the normal position below the floor. Toward the portal end of the section, the shallow depth of cover over the structure entailed the transition of



(*a*) MANHATTAN STRUCTURE

(*b*) QUEENS STRUCTURE

Fig. 5.—Steel Bent and Concrete Land Sections

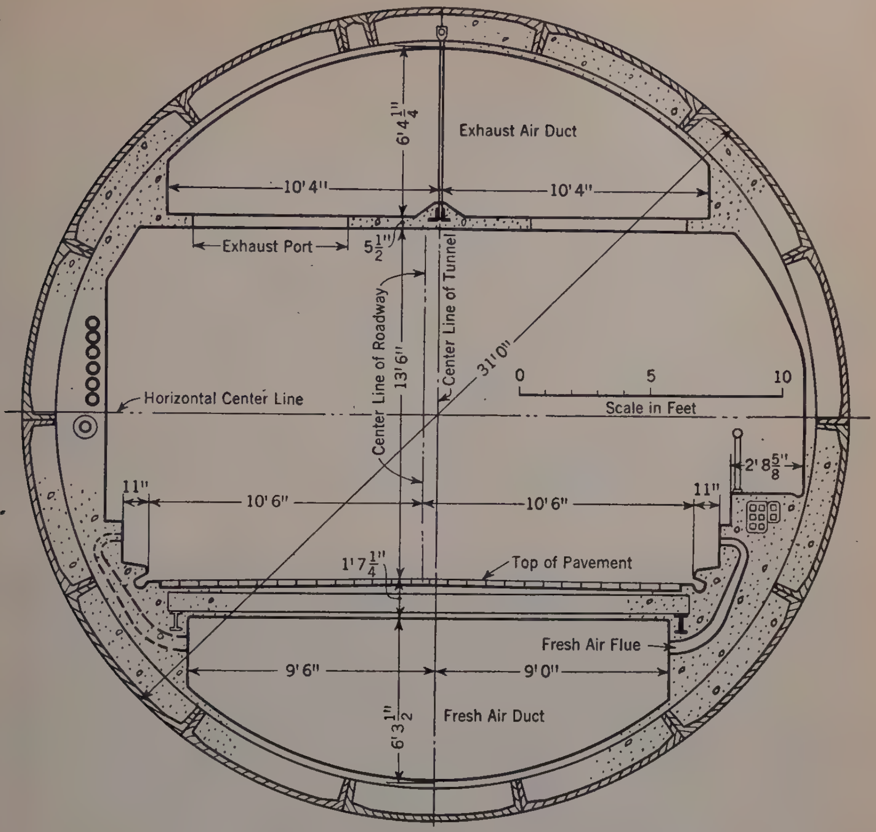


FIG. 6.—TYPICAL CAST-IRON-LINED TUNNEL SECTION IN EARTH

both the fresh air ducts and the exhaust ducts from above the ceiling to a location between the tubes.

Tunnel Interior.—The interior features of each tube include: (1) An interior concrete lining in the metal-lined sections of the structure; (2) roadway slab and paving; (3) a walk on the left of the direction of travel, approximately 2 ft 8 in. wide and 3 ft 6 in. above paving level; (4) a fresh air duct generally under the roadway, connected to the roadway space by fresh air flues; and (5) an exhaust air duct, generally above the ceiling, connected to the roadway space by openings in the ceiling. Fig. 6 shows a typical tunnel section.

The roadway slab consists of transverse 10-in. **I**-beams spaced 21 in. on centers with longitudinal reinforcing rods above and below, and transverse reinforcing bars between the **I**-beams. The roadway slab is designed in accordance with the theory advanced by H. M. Westergaard,[3] M. Am. Soc. C. E., in 1930. The roadway beams rest on concrete benches and when placed over fresh air flues are supported on 8-in. I-beams which span the flues. The ceiling slab, containing the exhaust ports, is a reinforced concrete slab 5.5 in. thick supported at the sides of the tunnel and hung at the center by stainless steel bars generally 10 ft 8 in. apart.

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[3] "Computation of Stresses in Bridge Slabs Due to Wheel Loads," by H. M. Westergaard, *Public Roads*, 1930, Vol. 11, No. 1, pp. 1–23.

The fresh air flues are not terminated in a continuous expansion chamber, as has been the practice heretofore, but discharge directly into the roadway space through openings 5 ft 4 in. long and 3 in. wide, formed in the glazed architectural terra-cotta blocks at the front of the benches (see Fig. 7). Experience has shown that the distribution of fresh air along the roadway can be just as efficient and continuous with direct discharge into the roadway space as through a continuous expansion chamber, particularly when the fresh air flues on either side of the roadway are staggered, and when the movement of traffic furnishes longitudinal distribution.

Clay ducts under the sidewalk are used for low-tension cables as, from this location, wiring to control boxes for signals, telephones, etc., was simpler than if low-tension conduits were placed in the opposite wall as has been the previous practice. Continuous lighting-cable conduits are placed in the walls of the upper air duct so that no conduits of any kind are contained in the ceiling slab.

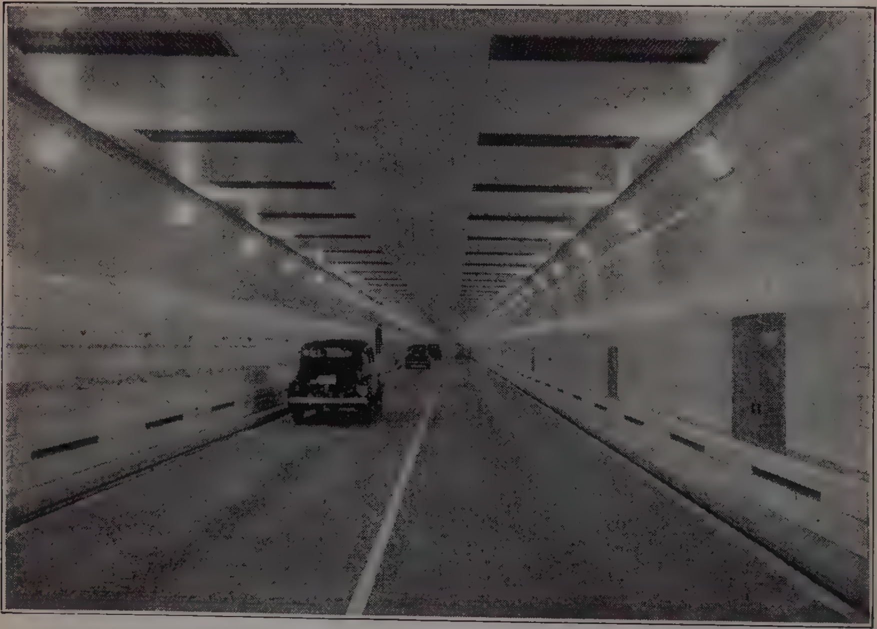


FIG. 7.—FINISHED INTERIOR OF TUNNEL

In each tunnel, between ventilation buildings, six 4-in. asbestos-cement conduits are provided in the wall opposite the sidewalk. The lower three conduits are so spaced as to be suitable for high-voltage transmission cables, and the upper three conduits are suitable for lower-voltage cables. The six conduits are available for rental, and are carried up the shafts to street-level splicing chambers in each ventilation building, where connections may be made to private transmission lines entirely independent of tunnel services.

Low-tension splicing chambers in the sidewalk bear a more or less constant relation to the police posts, which house the telephones and manual signal controls. These posts are spaced at such intervals as to provide visibility and easy access to the portions of the tunnel between them, in general about 250 ft. Two small niches, each holding one 15-lb carbon-dioxide fire extinguisher, are in the wall above the sidewalk at each control station and two additional small fire extinguisher niches are installed midway between control stations. This spacing makes fire extinguishers readily available to the tunnel policeman on duty at the control station. Larger fire extinguisher niches, each niche containing three 15-lb carbon-dioxide fire extinguishers, are installed in the wall on the side opposite the sidewalk. These niches are spaced at approximately 250-ft intervals for use by the tunnel police or by others. Niches containing 100 ft of racked fire hose attached to a $2\frac{1}{2}$-in. valved outlet from the 6-in. tunnel water service pipe are spaced about 125 ft apart. The chemical fire fighting equipment distributed through the tunnel (aggregating 350, 15-lb carbon-dioxide fire extinguishers) is ample for all small fires and is sufficient for the control of larger fires until such time as the emergency wrecking tractor and the police car (each of which is equipped with a foam generator type of fire extinguisher), which answer all fire calls, can arrive on the scene.

Other niches in the tunnel are:

Niches	Spacing (in feet)
Tools	500
Gate valve and clean-out for shutting off and cleaning out the 6-in. water service line	500±
Access to valves controlling outlets from the 6-in. water service pipe to the fresh air duct to flush the latter	500±
High-tension splicing chambers	460±

Roadway and ceiling manholes are spaced about 500 ft apart.

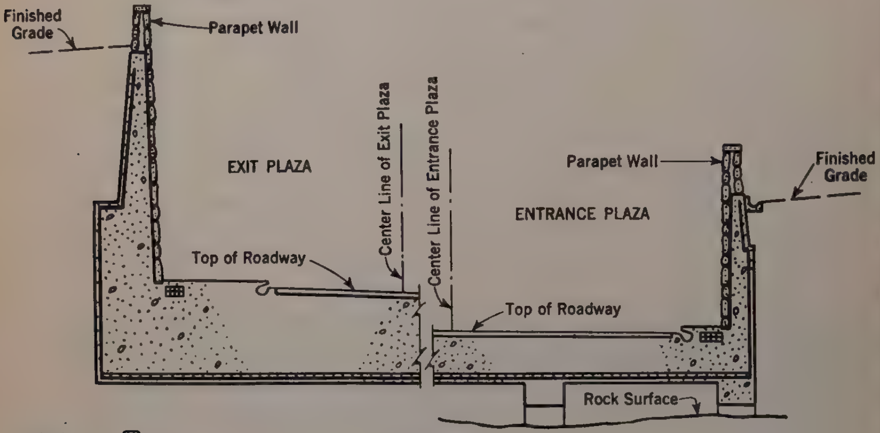
The tunnel roadway is paved with de-aired vertical fiber brick $3\frac{1}{2}$ in. in depth, laid transversely on a $\frac{3}{4}$-in. bituminous mastic cushion. Joints between bricks are filled with pure asphalt with 20% to 30% inert fine material added. The center line marker—a single continuous line on tangent and slight curves, and two continuous white lines about 8 in. apart on the sharper curves—is made with pure white-body vitreous bricks with hard bright, white glazed top surfaces and laid longitudinally end to end. Wherever double-lane markers are used, electrically illuminated amber prisms, protected by cast-steel hoods, are installed in the paving between the lanes at regular intervals.

The roadway surface is crowned 1 in. on tangent and superelevated on the outside of all curves. On curves of about 500-ft radius, the superelevation is a maximum of $\frac{3}{4}$ in. per ft. The transition from crowned to banked roadway is at a rate which will maintain a gutter grade of not less than 0.5% nor more than 6%. In the circular tunnel sections the side-walls are normal to the transverse slope of the banked roadway; and, in the steel bent and concrete sections, the side-walls are vertical and side clearances are increased as necessary.

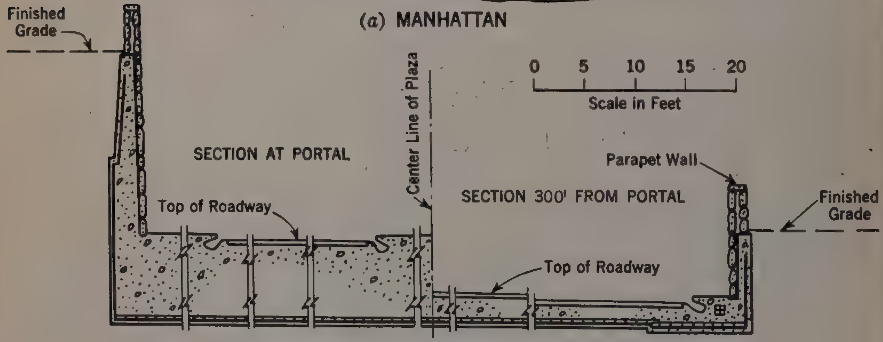
The walls of the tunnel are finished with ivory-tinted, ceramic, vitreous, semi-mat glazed tile with a diffused light reflection factor of 0.78. There is a continuous border of ming green tile at the ceiling.

The tunnel ceiling is finished with 6-in. by 6-in. light ivory-tinted pebbled glass tile set in copper alloy grippers in the ceiling slab.

The tunnel sidewalk bench is faced with light, buff-tone, mottled glazed, architectural terra cotta. A sidewalk railing 2 ft 3 in. high is composed of cream-colored, porcelain enameled steel pipe $1\frac{1}{2}$ in. in diameter.



(*a*) MANHATTAN



(*b*) QUEENS

FIG. 8.—TYPICAL PLAZA CROSS SECTIONS

The recessed light boxes for tunnel lighting have bronze fronts, and all niche and accessory doors are of commercial and extruded bronze shapes. All finish and hardware in the tunnel are flush, and signals are streamlined so as to facilitate washing and cleaning operations.

Plazas and Approaches, Manhattan.—Two types of design were studied for the Manhattan plazas. One type employed a drainage system under a comparatively thin invert slab, through which the underground water would be conveyed to a main drainage gallery and from there into the sumps to be removed by pumping. The other type of design utilized the weight of walls and heavy invert slab to counterbalance hydrostatic uplift, and, as construction costs would have been about the same in either case, this latter design was adopted as the more reliable.

In general, the typical cross section of each plaza is that of a **U**-shaped reinforced concrete structure in a waterproofing envelope with concrete protection outside the waterproofing (see Fig. 8). Invert slabs vary in thickness from 8 ft to 1 ft; the retaining wall height above the roadway is a maximum of 22.5 ft at the entrance portal and 31 ft at the exit portal. Invert subgrade is partly in rock cut and partly above rock surface elevations. In practically all cases, retaining walls were extended down to rock. For an area of the entrance plaza under which the overburden of rock contained layers of peat, the invert slab is supported on piers carried down to the rock. The invert slabs of both plazas are divided by expansion joints placed at 50-ft intervals; these joints are filled with premolded cork filler and provided with copper water stops.

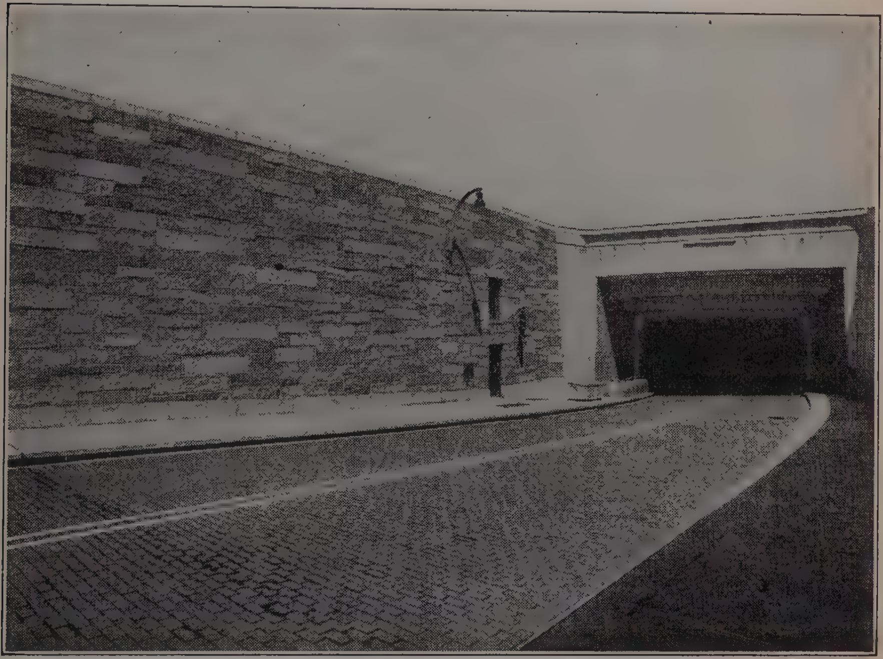


Fig. 9.—Plaza Wall Finish at Manhattan Entrance

The exit plaza underpass at Second Avenue is of the steel bent and concrete subway type of structure, as is the underpass under 36th Street.

The north and south tunnel entrance and exit avenues in Manhattan provide 32-ft roadways and a sidewalk generally 4 ft 4 in. wide on the side to the right of the direction of travel of each roadway. When practicable, the foundation walls of the demolished improvements were utilized for foundations of the parapet walls. In other instances the walls had to be built to retain high ground of adjacent areas.

Plazas and Approaches, Queens.—The Queens plaza may be considered structurally as consisting of two sections. The west section between the portal at Vernon Boulevard and 11th Street, where the roadway comes to grade, is similar in design to the Manhattan plazas. The east section of the plaza, starting at 11th Street and continuing to 21st Street and 50th Avenue, is constructed on compacted backfill contained by L-shaped reinforced concrete walls located on the south side; the roadway pavement is a reinforced concrete slab 10 in. thick laid directly on the backfill.

The east section of the plaza includes a ramp leading to the elevated express highway. Because the bearing capacity of the soil was found to be unreliable, the support for the last span of the highway connecting to the ramp consists of two 5-ft reinforced concrete cylinders bearing on rock at a depth of about 40 ft.

Plazas and Approaches, Architectural Finish.—The retaining walls of the Manhattan and Queens plazas are faced with coarse-texture, seam-face, granite, random ashlar; 60% golden-brown seam face, 20% bright seam face, and 20% split face. The lengths of the individual stones are as much as 8, and even 10, times the height, emphasizing the horizontal lines of the plazas (see Fig. 9: Plaza Wall Finish).

The tunnel portals are of six-cut Moose-a-bec granite with polished granite cheeks extending 20 ft into the tubes, and finishing flush with the tunnel tiling. The Queens portal is a twin design with exceptionally large pieces of granite. The soffits of the Manhattan portals are arched. Parapet walls are solid seam-face granite with six-cut granite copings.

The flood lights are of commercial type, placed at a height of 60 ft above the roadway and resting on granite pedestals.

Buildings.—The ventilation buildings of the Queens Midtown Tunnel (see Fig. 10) are classified as industrial power house structures. They are designed with structural steel skeleton frames and are of fireproof construction throughout.

A distinct departure from previous designs was the development of masonry-slotted jambs for fresh air louver blades at a substantial saving in cost over the all-bronze louver construction. The louver blades are ebony asbestos cement slabs impregnated with waterproof compound of fluxed natural asphalt.

The Manhattan ventilation building is 110 ft by 144 ft and is 124 ft high to the top of the two exhaust, or evasé stacks, which rise 14 ft above roof level. The building, which has an octagonal shape, is partly supported on the Manhattan ventilation shaft structure and partly on piers based on rock. This building houses twelve fresh air supply fans and twelve exhaust fans, a large switchboard room, transformer compartments, carbon-monoxide analyzer room, chemical room, maintenance rooms, electrical work shop, pressure-tank room, and elevator motor room. Two enclosed stairways and an elevator in the building provide access from the tunnel roadways to all floors. The cubic-foot cost of the building was $0.439, not including the cost of foundations or rough first floor slab.

The Queens ventilation building, which is partly supported on the bent structure of the tunnel and partly on concrete-filled steel cylinders, is 107 ft by 113 ft in plan with a height of 121 ft to the top of the evasé stacks, which rise about 30 ft above the roof level. Its structural and architectural treatments are similar to those of the Manhattan building. Eleven fresh air supply fans and eleven exhaust fans are housed on three floors, and other equipment and facilities are as for the Manhattan building. Exclusive of foundations and rough first floor slab, the cost of this building was $0.404 per cu ft.

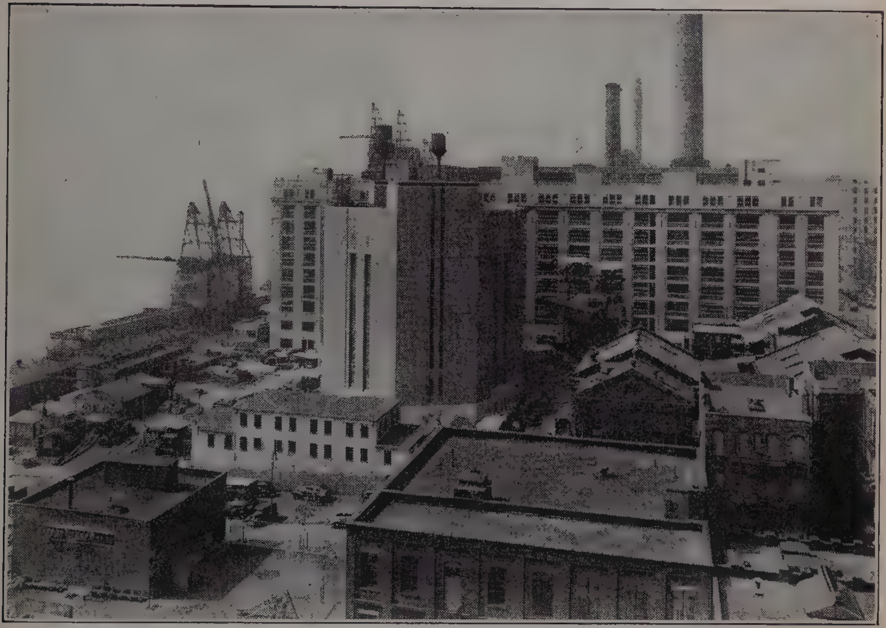


Fig. 10.—Manhattan Ventilation Building, Nearing Completion

The service building in Queens is approximately 100 ft by 185 ft in plan, and is a single-story structure housing shops and a garage for maintenance equipment and two stories and basement housing the rooms for supervisory control, electrical equipment, police quarters, detail room, and maintenance and operating offices. The cubic-foot cost of this building was $0.543.

Tunnel Ventilation

The method of ventilating the Queens Midtown Tunnel follows closely that developed for the Holland Tunnel. A continuous supply of fresh air is introduced to the driveway just above the curb through fresh air flues generally spaced at intervals of 10 ft 8 in. on upgrades and level grades, and 16 ft on downgrades, on each side of the roadway, locations on opposite sides of the roadway being staggered. These fresh air flues are fed from a longitudinal supply duct under the roadway. The vitiated air is removed through exhaust air ports placed at similar intervals in the roadway ceiling slab, above which is the longitudinal exhaust air duct.

Each tunnel is divided into four separate ventilating sections. With the exception of a short ventilating section on the Queens end of the north tunnel, which has two fresh air fans and two exhaust air fans, all the ventilating sections are connected to three fresh air fans and three exhaust air fans.

For normal maximum ventilation requirements, fifteen fresh air supply fans and fifteen exhaust air fans are operated. The remaining sixteen fans are stand-by spares.

The fans are driven by totally-enclosed, fan-cooled, squirrel-cage, induction motors through silent chain transmissions. Each fan on the downgrade entrance sections in Queens and Manhattan is driven by a single two-speed motor. All the other fans are driven by a dual motor drive consisting of one large single-speed motor to operate the fan at full speed and a small two-speed motor to drive (through a chain drive to the large motor) the fan at $66\frac{2}{3}\%$ and at $33\frac{1}{3}\%$ of full speed. With this combination of fan speeds and number of fan units, greater economies of operation have been accomplished than were possible at the Holland Tunnel where the fans are driven by slip-ring motors of the wound rotor type with external resistance for varying the speed.

TABLE 1.—COMPARISON OF VENTILATION; QUEENS MIDTOWN AND HOLLAND TUNNELS

(Twin Tunnels in Each Case)

No.	Description	Holland	Midtown
	Length of Tunnel, in Feet:		
1	North	8,557	6,414
2	South	8,371	6,272
3	Total ventilated	16,928	12,686
	Number of:		
4	Ventilation sections	14	8
5	Ventilation equipment, buildings	4	2
6	Fresh air fans	42	23
7	Exhaust air fans	42	23
8	Electric motors	84	82
9	Air changes per hour	42	42
	Volumes, in Cubic Feet per Minute:		
10	Fresh air supplied	3,765,200	2,889,000
11	Air exhausted	3,915,600	2,954,000
	Fan Brake Horsepower Installed:		
12	Total	6,260	4,118
	Normal Maximum Operating Fan Brake Horsepower:		
13	Total	4,173	2,725

During 1941 when approximately 4,400,000 vehicles were carried by the tunnel, of a total electric power bill of $33,412.35 (which was mostly for lighting), only $4,704.30 was for ventilation, or a rate of about 0.1¢ per vehicle. Table 1 is a comparison of the salient features of the ventilating system of the Holland Tunnel and the Queens Midtown Tunnel.

TUNNEL LIGHTING

The tunnel is lighted by means of individual luminaires along the walls of the tunnel, immediately below the ceiling. Each luminaire contains a 150-watt and a 100-watt incandescent lamp on independent circuits. Three levels of illumination are used according to the intensity of daylight outside the tunnel. Additional illumination during the daytime inside the entrance portals is provided by 1,000-watt and 500-watt enclosed fixtures suspended in open bays in the ceiling of the tunnel, graduated from five 1,000-watt fixtures per bay at the portal to three 500-watt fixtures per bay at 160 ft inside the portal. The graduation of light at the portals, produced by this additional illumination in combination with the three levels of tunnel illumination, has eliminated the discomfort and blinding effects experienced by motorists on entering other vehicular tunnels.

Lighting circuits are arranged so that a motorist is always within the light of luminaires fed from four sources of power regardless of whether the 150-watt lamps or the 100-watt lamps are burning.

TUNNEL OPERATION

The central point of control of operation is in the service building, which contains the electrical control board from which ventilation fans are started and stopped and their speed changed, from which the traffic signals and alarms for both fires and wrecks are piloted and controlled, and from which the intensity of tunnel lighting is varied.

The board also carries the carbon-monoxide recorders of the various ventilation sections of the tunnel and indicators of the rate of flow of vehicles in each tunnel.

The personnel attending to the movement of vehicles functions as a private uniformed police force supervised by a captain who reports to the superintendent. The mechanical, electrical, and structural maintenance force reports directly to the superintendent. The operating staff was organized under the direction of the chief engineer and, after six weeks of successful operation, was turned over to the direction of the administration department.

The arrangement of the organization, which was an innovation over other tunnel projects, has proved itself to be sound and economical. The cost of operation during 1941 was $587,916.22, or less than 40% of the cost of operating the Holland Tunnel.

CONSTRUCTION

Construction Program and Costs.—The construction work of the project was divided into thirty-one contracts, including three for test borings; and, in addition, there was a miscellaneous group of contracts and open-market orders for operating equipment. A coordinated program for preparation of plans, contracts and specifications, advertising, opening bids, awarding, and times for completion of these contracts, was prepared when work began on the project. This schedule, of course, was subject to some modification and revision from time to time to suit varying requirements and conditions as the work progressed, but the essential principle of the program was successfully adhered to—that is, to complete the construction of the project in the minimum time practicable, with an accumulation of payments to contractors for the work as completed, which would result, as nearly as possible, in a minimum total for interest during construction.

All work was let under open competitive bidding and the contracts were awarded to the lowest responsible bidder. In all cases, this was the low bidder, with the exception of contract No. 2-A for borings at the site of the Manhattan ventilation shaft where the low bidder was unable to start work as ordered because of inability to obtain labor of the required type.

Table 2 gives a list of the contracts for the project, together with final costs, and the dates of beginning and completing the field work. The dates for completion are the actual dates when all work under each contract was fully completed and accepted, in some cases extending beyond the date of opening the tunnel.

The first construction contract to be let was No. 1, and the second, No. 3. The individual letting of these contracts permitted work to progress on them while plans, contract, and specifications were completed for contract No. 4, the principal contract of the project. When contract No. 4 was let, the construction shaft in Queens had been completed and was ready to serve as the point of operations for tunneling from Queens; tunneling operations from Manhattan were started in the ventilation shaft excavated under contract No. 3 and, later, were transferred to construction shafts sunk to the west of the ventilation shaft, thus permitting work on the Manhattan ventilation building to proceed. With the letting of contract No. 7, the entire main structure of the project from portal to portal was placed under construction. All remaining contracts were coordinated to these three. Work on the Manhattan ventilation building was suspended from August 14, 1939, to November 1, 1939, and on the Queens ventilation building from February 17, 1940, to May 27, 1940, while ventilation equipment was being installed.

TABLE 2.—TIME OF START AND FINISH AND COST OF CONTRACTS ON THE QUEENS MIDTOWN TUNNEL

Contract No.	Description of work	Started	Finished	Total cost
1	Queens construction shaft	Oct. 5, 1936	June 9, 1937	$ 201,074.06
2	Land and river borings	Oct. 13, 1936	Jan. 13, 1937	49,585.17
2A	Manhattan shaft borings	Oct. 2, 1936	Oct. 10, 1936	3,644.57
2B	Manhattan plaza borings (modified to include Queens plaza borings)	Oct. 4, 1937	Nov. 30, 1937	4,477.55
3	Excavation for Manhattan ventilation shaft	Jan. 4, 1937	July 29, 1937	299,332.34
4	River tunnels and shafts	July 14, 1937	June 3, 1941	22,133,304.32
4A	Dredging and pile driving	Sept. 14, 1938	Nov. 17, 1938	63,449.60
5	Manufacture, storage, and delivery of cast-iron tunnel lining	Jan. 4, 1937	Oct. 31, 1939	3,333,587.58
6	Manufacture, storage, and delivery of bolts, nuts, and washers	Mar. 8, 1937	Sept. 19, 1939	191,781.14
7	Queens land section	Apr. 25, 1938	Aug. 21, 1940	2,173,613.45
8	Manhattan land section	June 8, 1938	Nov. 14, 1940	3,387,484.07
9	Manhattan ventilation building	Sept. 12, 1938	Sept. 30, 1940	658,000.00
10	Queens ventilation building	May 26, 1939	Dec. 12, 1940	548,093.66
11	Manufacture and installation of fans, motors, and transmissions	Dec. 22, 1937	Dec. 18, 1940	284,689.00
12A	Manhattan plazas	July 24, 1939	May 13, 1941	1,870,128.95
12B	Manhattan surface approaches	Mar. 7, 1940	May 15, 1941	257,706.28
12C	Demolition of buildings for Manhattan plazas	Mar. 7, 1939	July 19, 1939	43,495.00
13	Manufacture, storage, and delivery of ceiling tile	Oct. 5, 1938	July 22, 1940	238,876.41
14	Queens plaza	May 15, 1939	Oct. 25, 1940	643,001.91
14A	Demolition of buildings for Queens plaza	Jan. 24, 1939	Apr. 22, 1939	16,821.00
15A	Electrical installation	Aug. 14, 1939	Feb. 13, 1941	584,077.60
15B	Manufacture, storage, and delivery of switchboards	Sept. 22, 1939	Oct. 9, 1940	427,750.00
15C	Electrical fixtures	June 20, 1940	Feb. 24, 1941	98,339.00
16	Tunnel finish	Mar. 25, 1940	Feb. 5, 1941	339,411.87
17	Pumping equipment	May 22, 1940	Dec. 11, 1940	46,120.00
18A	Toll booths	June 11, 1940	Feb. 14, 1941	90,425.44
18B	Queens plaza finish	June 19, 1940	Jan. 10, 1941	88,154.48
18C	Manhattan portal finish	Aug. 22, 1940	Dec. 30, 1940	73,141.76
18D	Manhattan plaza finish	Nov. 20, 1940	June 19, 1941	249,196.91
19	Service building and Queens emergency garage	May 23, 1939	Oct. 30, 1940	296,911.36
20	Tunnel and plaza paving	July 12, 1940	Dec. 10, 1940	193,026.67
	Total contract costs	....	....	$38,888,701.38

Work on the site of the project was conducted under the rules and regulations of the PWA specifying minimum wages and maximum hours of work—the 30-hr week, applying to contracts Nos. 1 to 3, inclusive, and, by special dispensation, the 40-hr week to contracts Nos. 4 to 20, inclusive.

Total contractors' labor for the project amounted to approximately 9,193,000 man-hr at the site, at an average hourly wage of $1.61. For contract No. 4 alone, the total man-hours were 5,942,000 and the average hourly wage was $1.78. These figures for man-hours and average hourly wages are based on the actual working hours and hourly wages paid, including the net work-shift hours of compressed-air workers whose wages were on a daily basis.

Shafts.—In Queens, tunneling operations for the river tunnels were conducted through a working shaft located on the center line of tunnels 260 ft east of the east building line of Second Street. The shaft was 86 ft 9 in. by 33 ft 9 in. and was excavated to subgrade depth at 67.4 ft below street surface. The shaft structure was of steel and concrete. Two octagonal openings, exposing the rock wall of the excavation, were provided on 45-ft centers in the west wall of the shaft through which to drive the tunnel shields.

The Manhattan ventilation shaft was excavated to net lines 60 ft 2 in. by 111 ft 8 in. in plan and an average depth of 112 ft below ground surface, the excavation being mostly in rock. The shaft was used for initial tunneling operations and, as a precaution during this period, lined with a designed minimum thickness of 4 in. of protective concrete reinforced with wire mesh, and the walls were braced temporarily with timber frames.

The two construction shafts in Manhattan, to which tunneling operations were transferred, were 25 ft by 25 ft in plan, and centered on the tunnel lines an average distance of 112 ft west of the center of the ventilation shaft. The shafts were excavated to a depth of 4 ft below the tops of the respective tunnels, and the rock headings from the ventilation shaft driven through them.

River Tunnels.—Construction of the river tunnels only will be discussed in this paper as the limitation of space will not permit any further discussion of the other contracts of the project.

In the New York area, three large diameter tubes comparable in size to those of the Queens Midtown Tunnel—Holland Tunnel (two tubes, 29.5 ft in diameter) and the south crossing of the Lincoln Tunnel (one tube, 31.0 ft in diameter)—had been driven successfully in the bed of the Hudson River. On the other hand, the experiences in tunneling under the East River, to 1937, had been gained mainly with rapid transit tunnels, 18 ft 3 in. or less in diameter, and with the four tubes for the Pennsylvania Railroad,[4] 23 ft in diameter. The latter provided the best available basis for comparison in anticipating construction conditions in the Queens Midtown Tunnel. Uncertainties in connection with driving the substantially larger 31-ft tubes through the porous overburden and varying rock conditions of the bed of the East River contributed to a narrowing of the field of competition for contract No. 4, for which only two bids were received. The Walsh Construction Company, low bidder, was awarded the contract and completed the work well within schedule.

Contract No. 4 extended between limits 4,086 ft apart in the south tunnel and 4,111 ft apart in the north and comprised all of the shield-driven, cast-iron-lined tunnel of the project (3,772 lin ft in the south tunnel and 3,756 lin ft in the north); part of the rock tunnel lined with 32 ft 10 in. structural steel

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[4] "The New York Extension of the Pennsylvania Railroad," *Transactions*, Am. Soc. C. E., Vols. LXVIII and LXIX (1910).

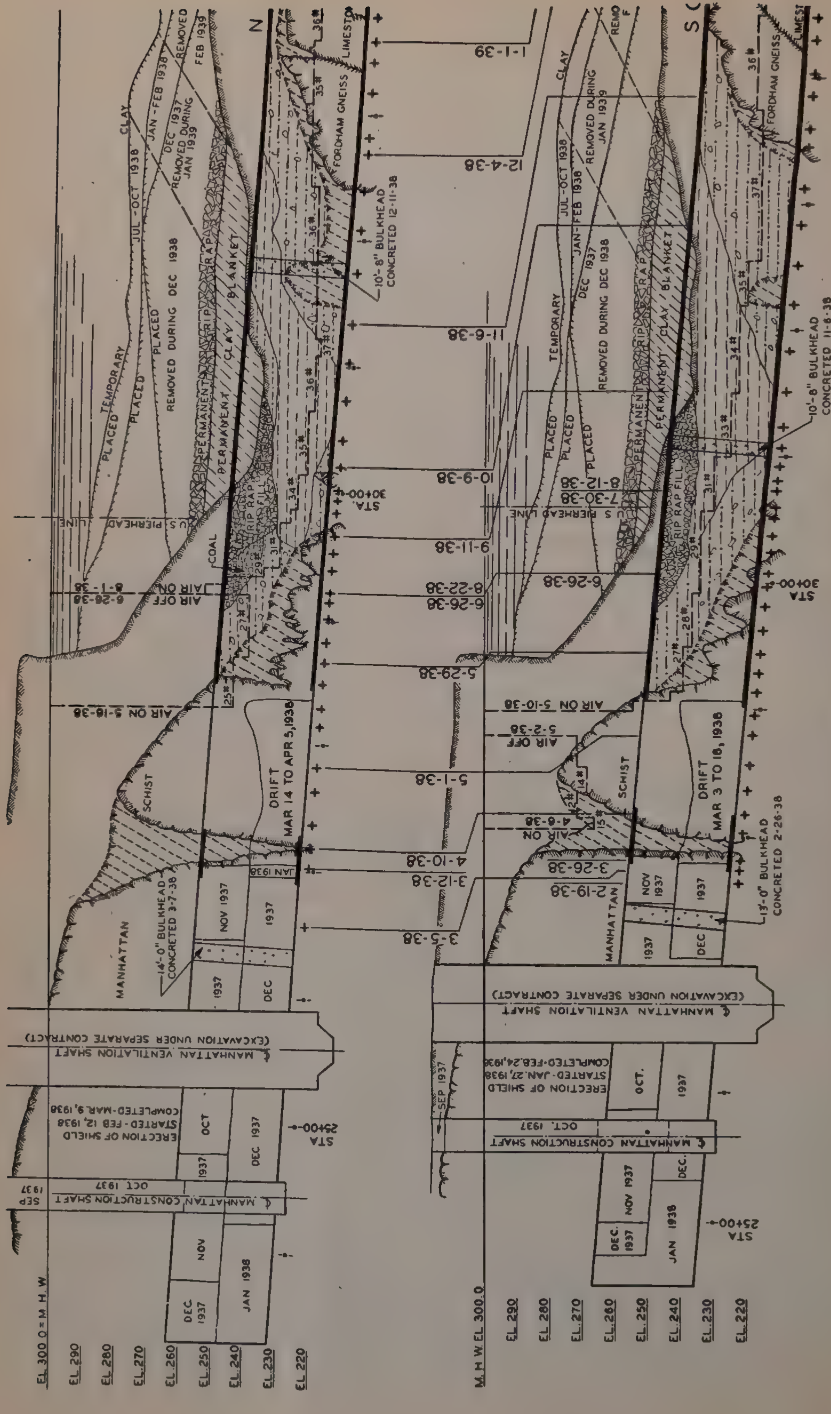


FIG. 11A.—LONGITUDINAL SECTIONS SHOWING PROGRESS OF RIVER TUNNELS; STATIONS 25+0 AND 30+0

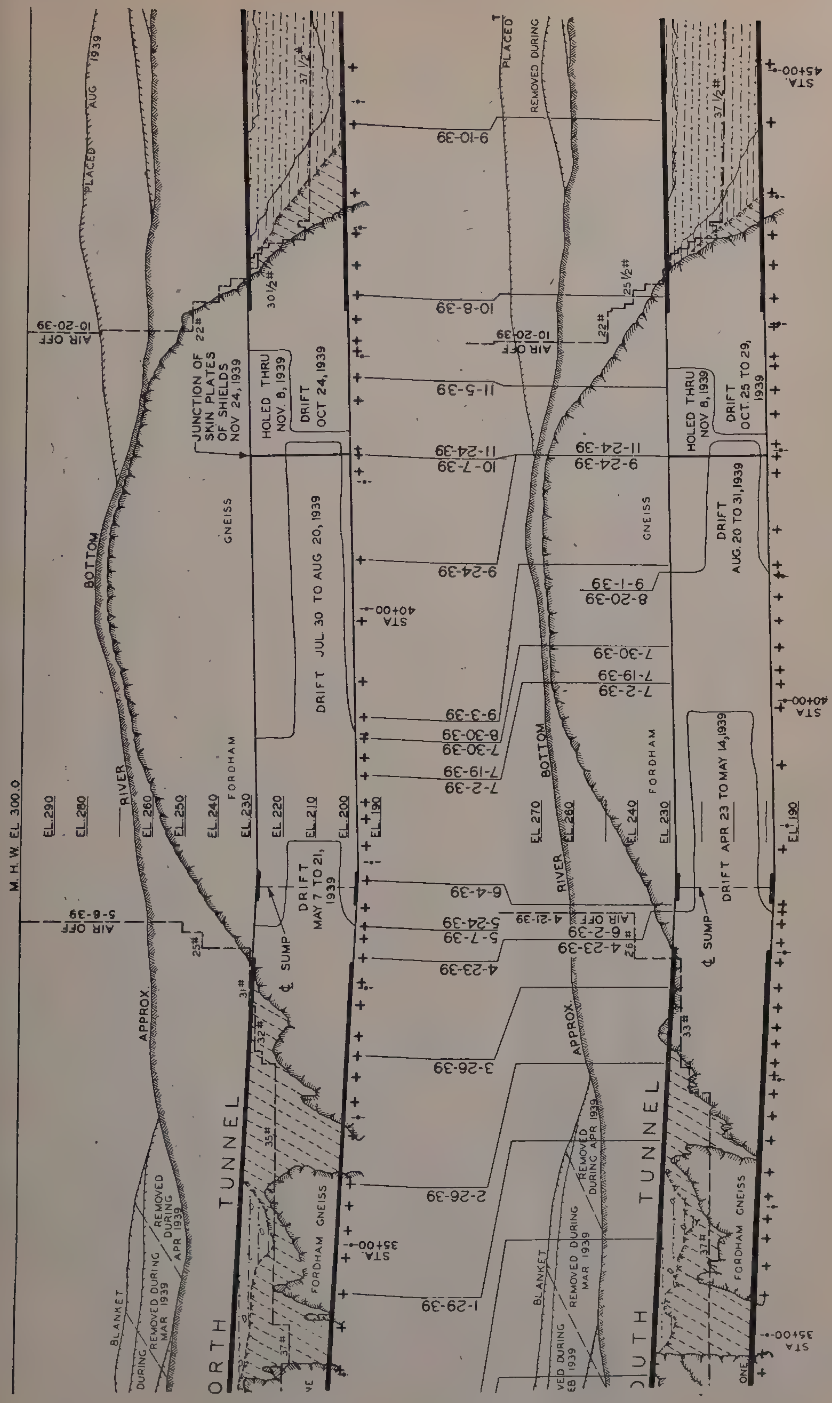


FIG. 11B.—LONGITUDINAL SECTION SHOWING PROGRESS OF RIVER TUNNELS; STATIONS 35+0 AND 45+0

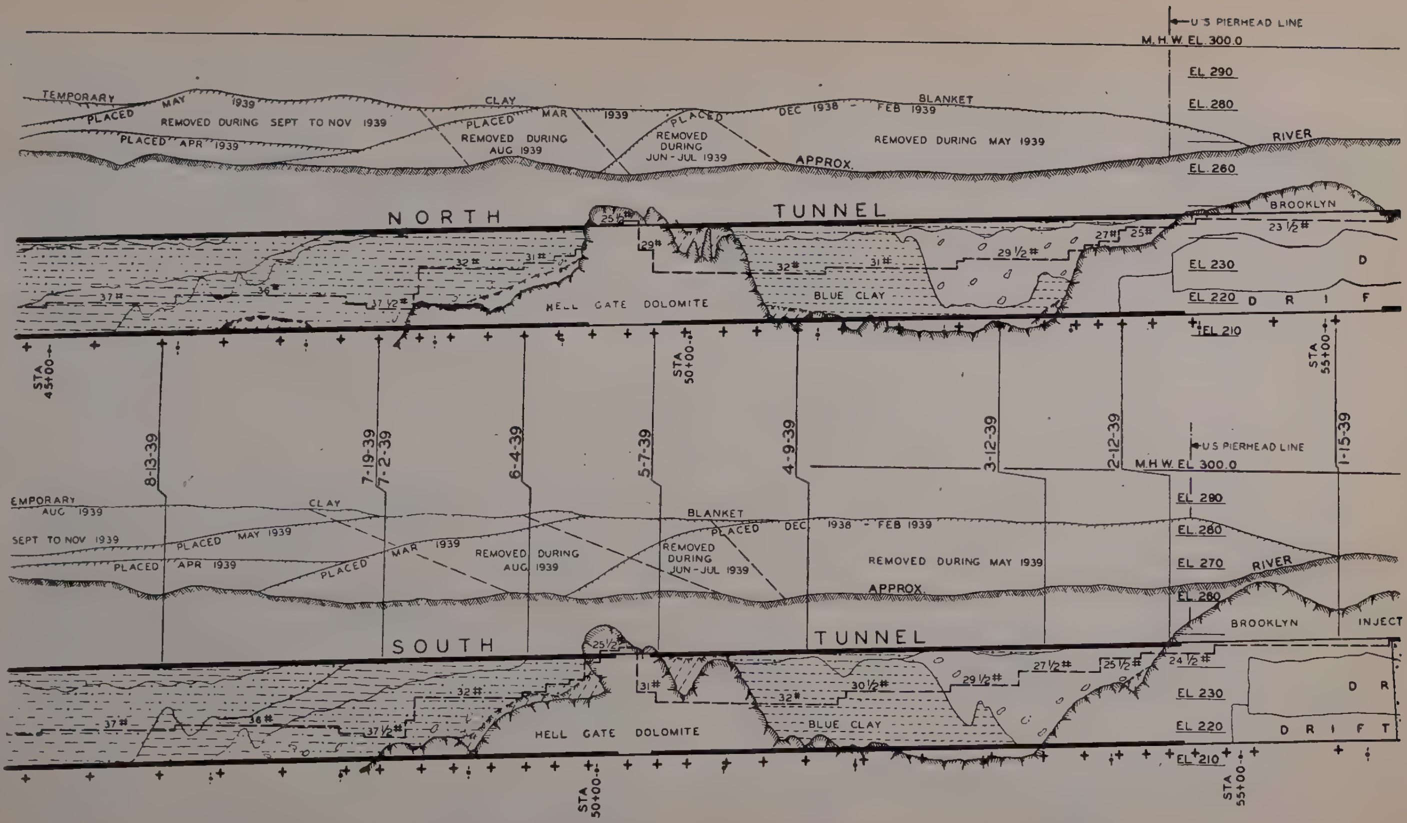


Fig. 11C.—Longitudinal Section Showing Progress of River Tunnels; Stations 50 +0 and 55 +0

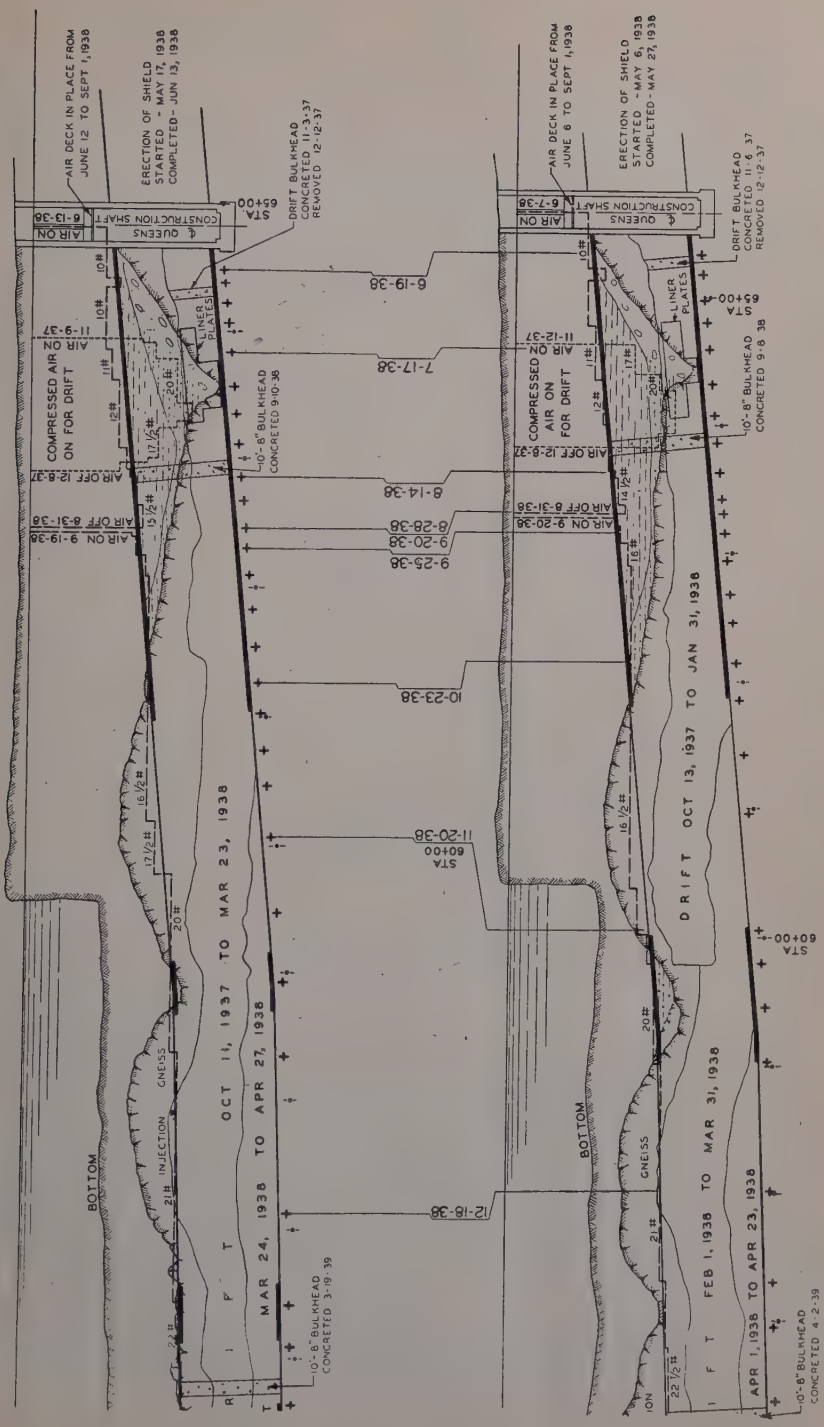
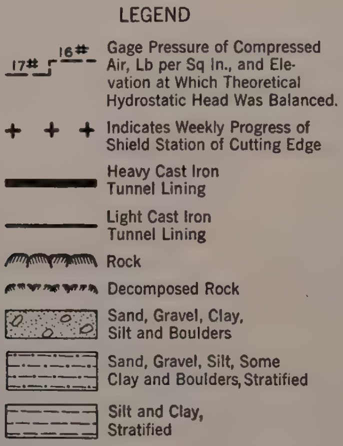

AIR DECK IN PLACE FROM JUNE 12 TO SEPT 1, 1938
ERECTION OF SHIELD STARTED - MAY 17, 1938 COMPLETED - JUN 13, 1938
AIR ON 6-13-38
℄ QUEENS CONSTRUCTION SHAFT
STA. 65+00
DRIFT BULKHEAD CONCRETED 11-3-37 REMOVED 12-12-37
10#
10#
11#
12#
17 1/2#
20#
15 1/2#
LINER PLATES
COMPRESSED AIR ON FOR DRIFT
AIR ON 11-9-37
AIR OFF 12-8-37
AIR OFF 8-31-38
AIR ON 9-19-38
10'-8" BULKHEAD CONCRETED 9-10-38
6-19-38
7-17-38
8-14-38
8-28-38
9-20-38
9-25-38
10-23-38
11-20-38
STA 60+00
12-18-38
DRIFT OCT 11, 1937 TO MAR 23, 1938
MAR 24, 1938 TO APR 27, 1938
16 1/2#
17 1/2#
20#
GNEISS
INJECTION
21#
22#
BOTTOM
10'-8" BULKHEAD CONCRETED 3-19-39
AIR DECK IN PLACE FROM JUNE 6 TO SEPT 1, 1938
ERECTION OF SHIELD STARTED - MAY 6, 1938 COMPLETED - MAY 27, 1938
AIR ON 6-7-38
℄ QUEENS CONSTRUCTION SHAFT
DRIFT BULKHEAD CONCRETED 11-6-37 REMOVED 12-12-37
STA 65+00
LINER PLATES
AIR ON 11-12-37
COMPRESSED AIR ON FOR DRIFT
AIR OFF 12-8-37
10'-8" BULKHEAD CONCRETED 9-8-38
11#
12#
17#
20#
14 1/2#
AIR OFF 8-31-38
AIR ON 9-20-38
16#
DRIFT OCT 13, 1937 TO JAN 31, 1938
16 1/2#
STA 60+00
20#
BOTTOM
GNEISS
21#
FEB 1, 1938 TO MAR 31, 1938
APR 1, 1938 TO APR 23, 1938
22 1/2#
ION
10'-8" BULKHEAD CONCRETED 4-2-39

FIG. 11D.—LONGITUDINAL SECTION SHOWING PROGRESS OF RIVER TUNNELS; STATIONS 60+0 AND 65+0

(225 lin ft in the south tunnel and 265 lin ft in the north); completion of the Manhattan ventilation shaft structure to surface line; and the necessary work in the Queens construction shaft. Prices for tunnel excavation east of the Manhattan ventilation shaft under this contract were as follows: Earth, \$62.00 per cu yd; rock, \$52.50 per cu yd; and mixed face, \$96.00 per cu yd. These are the highest unit prices for tunnel excavation in any corresponding quantities that have come to the writer's attention and reflect the uncertainties and difficulties anticipated in connection with driving such large-diameter tubes under the adverse ground conditions of this part of the East River.

Figs. 11 show progress in tunneling, together with materials penetrated, air pressures, point of balance of hydrostatic head, and sequence of placing and removing clay blanket. Weekly progress is shown for the advance of the shields. The junction between steel and cast-iron lining is 115 ft east of the Manhattan ventilation shaft in the north tunnel and 85 ft east in the south. The legend for Figs. 11 is as follows:


LEGEND
16#
17#
Gage Pressure of Compressed Air, Lb per Sq In., and Elevation at Which Theoretical Hydrostatic Head Was Balanced.
Indicates Weekly Progress of Shield Station of Cutting Edge
Heavy Cast Iron Tunnel Lining
Light Cast Iron Tunnel Lining
Rock
Decomposed Rock
Sand, Gravel, Clay, Silt and Boulders
Sand, Gravel, Silt, Some Clay and Boulders, Stratified
Silt and Clay, Stratified

Note: Elevation 300.00 is the Datum of the New York City Tunnel Authority Which is 2.653 Feet Above Mean Sea Level at Sandy Hook, N.J.

In Manhattan, the procedure was to drive the rock headings from the ventilation shaft (Fig. 11A), west to the limits of the contract and east to the beginning of cast-iron-lined tunnel. The shields were then erected and equipped at the bottom of the ventilation shaft and advanced in the excavated headings a sufficient distance to permit construction of the tunnel bulkheads. With the tunnel bulkheads in place, bottom drifts were excavated in free air as far as rock cover would permit. Then, the shields were advanced completing the full face of excavation and erecting the permanent cast-iron lining, compressed air being placed on the headings when test drilling encountered soft material. Operations were transferred to the construction shafts on March 31, 1938.

From the Queens side (Fig. 11D), bottom drifts were advanced from the construction shaft for 1,119 ft in the north tunnel and 1,056 ft in the south tunnel (to about Station 55+0, Fig. 11C), where the rock floor dropped off in elevation rapidly. Compressed air was required in short stretches in each drift just west of the construction shaft, because of a dip in the rock floor there, the soft ground in the upper part of the drift being supported by liner plates. The remaining lengths of the drifts were excavated in free air and driven as high as safe rock cover would permit. The drifts were excavated to subgrade and concrete cradles were constructed on which to advance the shields, which were erected and equipped in the construction shaft. Shield driving, excavating full section, and erecting cast-iron lining were started under air pressure, work being conducted through an air deck placed on the construction shaft. When the shields were advanced sufficiently, the headings were bulkheaded with 12-in. concrete walls placed against the breasting, air pressures were reduced to normal, the shaft air deck was removed, the tunnel bulkheads and locks were placed, air pressure again was placed on the headings, and shield driving was resumed.

Excavation from the Manhattan side was stopped in the south heading on September 26, 1939, and in the north heading on October 7, 1939. Excavation from the Queens side was continued until the holing through in both headings on November 8, 1939. The shields were brought together and all but the steel skins removed, closure rings erected, tunnel lining cleaned and calked, and the interior concrete lining placed, progressing with concreting from Manhattan to Queens.

It was necessary to deposit a temporary clay blanket on the bed of the river in both the west and east channels, in advance of the headings. The riprap shoulders of the permanent blanket in the west channel were placed first. When tunneling was completed, all temporary clay was removed, and dredging was carried down so as to permit placing 5 ft of riprap at required channel depths to complete the permanent blanket.

Altogether 289,330 cu yd of material were placed in the west channel. Some of this was dredged and reused in the east channel, where the total clay deposited amounted to 125,000 cu yd. The blanket was approximately 140 ft wide at the top, with slopes of 1 on 4, or flatter at the sides, averaging about 285 ft in width, toe to toe, with a maximum of 480 ft.

Each of the four tunnel shields was 31 ft 8 in. in outside diameter, the lower half having an over-all length of 15 ft 10 in. from end of tail to cutting edge. Fig. 12 is a view of the shield for the south tunnel, Manhattan. Above the horizontal diameter at the front end, the hood projected 2 ft 10 in., making the upper half of the shield 18 ft 8 in. long. The inside diameter of the tail of the shield was 31 ft 2 in., providing a 1-in. clearance all around the outside of the 31-ft tunnel lining. The thickness of tail was 3 in., and the clearance from the face of the shoving jacks to the end of the shield was 6 ft 4 in.

Ring girders, 5 ft $2\frac{3}{4}$ in. on centers, and horizontal and vertical struts formed the intermediate section of the shield which was divided into thirteen pockets. Above the springing line there were eight working pockets with sliding platforms, each of which was operated by a 96-ton jack. Below the springing line a large center pocket 10 ft $9\frac{7}{16}$ in. high and 8 ft 8 in. wide permitted access of a mucking machine to the face of excavation. The upper two of the remaining four pockets were provided with sliding platforms and jacks as were those above the springing line. Each shield was equipped with sixteen hydraulic face jacks, twelve of 50-ton capacity and four of 96-ton capacity; and there were two 96-ton hydraulic jacks carrying a waler beam for bracing the face at about midheight of the center bottom pocket. Twenty-eight hydraulic jacks 10 in. in diameter were provided for shoving the shield, generally operated at a pressure of 5,000 lb per sq in., and a hydraulic erector arm was provided at the axis of the shield for erecting the lining segments. The face jacks and sliding platforms of the shields were effective in supporting the face of excavation under very difficult conditions, while the large bottom pocket permitted machine mucking in all materials encountered.

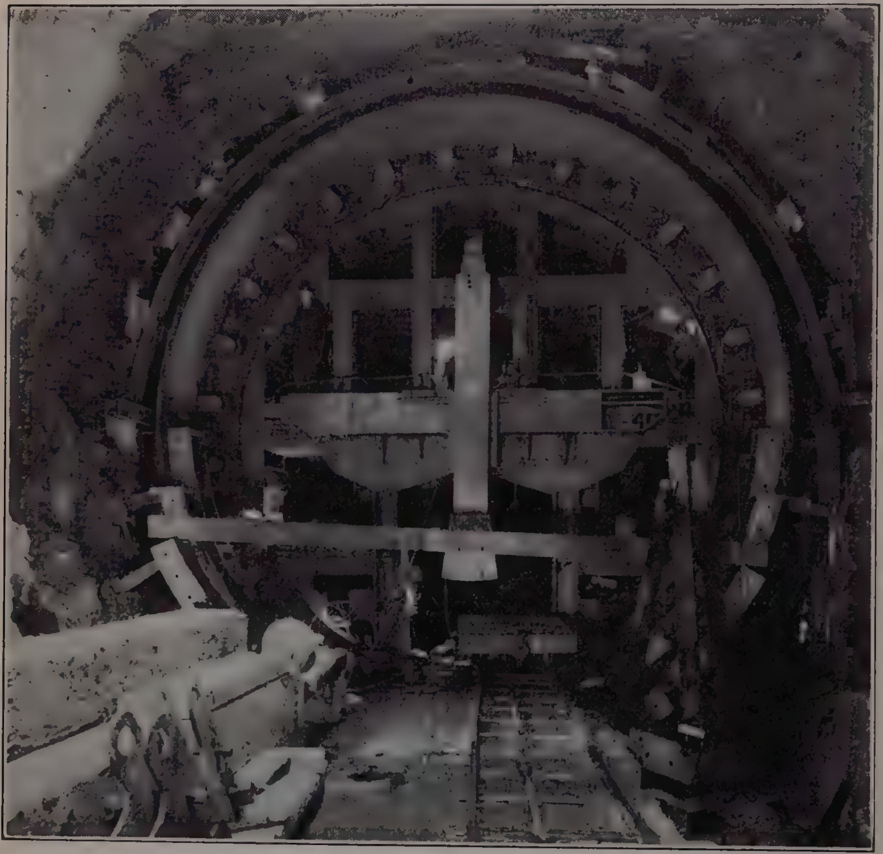


Fig. 12.—Rear View of Shield, South Tunnel, Manhattan

Tunneling operations were generally conducted on a 24-hr-per-day basis, six days per week.

Whenever practical, bottom drifts were driven in rock in advance of the shields (see Figs. 11). Bottom drifts on the Manhattan side ranged in size from

15 ft wide by 8 ft high to 22 ft by 22 ft; in Queens, it was possible to enlarge the drifts in advance of the shields to full section in some instances.

The tunnels were supplied with low-pressure, compressed air through 14-in. pipes, extending in duplicate from the compressor plants on the surface to each heading. The equipment of the Manhattan compressor plant was typical of both plants and consisted of seven low-pressure air compressors with a total capacity of 40,930 cu ft per min of free air, and three high-pressure air compressors with a total capacity of 6,000 cu ft per min. The high-pressure supply could be used to supplement the low-pressure supply and in the specifications—which required a minimum low-pressure air capacity of 45,000 cu ft per min of free air at a gage pressure of 50 lb per sq in., for each side of the river—80% of the high-pressure air capacity was allowed in computing the low-pressure air capacity of the plant. Three independent sources of power were provided in Manhattan with three separate 13,800-volt electric feeders. In Queens, there were three separate feeders of 27,000 volts each. Adequate capacity and continuity of supply in the low-pressure compressed air plants were vital factors in the successful completion of tunneling operations.

Operations in mixed face consisted, in general, of excavating and breasting the face at the plane of the front edge of the hood down to rock surface. The breast boards were 2-in. or 3-in. planks and were supported by vertical "soldiers," face jacks, and sliding tables. The rock face was excavated sufficiently by drilling and blasting to permit the cutting edge at the bottom of the shield to advance a ring length. If rock extended above the springing line, it was drilled and shot out around the hood just before making the shove. When there was no bottom drift, the shield cradle was made up of concrete-filled burlap bags.

In full face of earth, depending on the nature of material in the lower quarter of the face, the breasting was extended down generally from 6 ft to 12 ft below the springing line, averaging about 7.5 ft. When boulders were encountered in the lower part of the face, the breasting had to be extended down almost to the bottom of the heading. The face was breasted down in the plane of the edge of the hood, and, when the shove was started, the top boards were gradually removed and the material broken down to permit the advance of the hood cutting edge.

The most unfavorable conditions were encountered on the Manhattan side where riprap and other very porous materials in the upper part of the face permitted the compressed air to escape in large quantities, particularly in the area just outside the river bulkhead, where, in order to avoid difficulties with piles, the wharf over each tube had been demolished, the piles pulled out, and a heavy clay blanket placed prior to tunneling. Here, the shields were being advanced in mixed ground, excavating full face with each shove. The consumption of compressed air became dangerously near the full capacity of the plant, including that of the high-pressure air supply. Power consumption was at the rate of approximately 70,000 kw-hr per day. (The power bill for the plant, under these conditions, averaged approximately $4,900 per week.) Air blows through the blanket in the area of the demolished wharf were serious, and so eroded the blanket as to require its constant replacement. The north tunnel was shut down from June 16, 1938, to June 21, 1938, and the heading grouted up with volcanic clay, but the loss of air upon resuming operations in both headings was as great as ever.

On Sunday, June 26, 1938, at 8 p.m., a fire of undetermined origin was discovered in the north heading on the Manhattan side. The north and south headings were about abreast at this time (Fig. 11A). The fire was extinguished by lowering the air pressure and thereby flooding the tunnel at 8:30 a.m., June 27. Before successfully recovering the heading, the material adjacent to and in front of the north shield was grouted through eight $2\frac{1}{2}$-in. casings driven from the river surface, using 4,157 bags of mortar cement in a 1 : 1 mix with beach sand. The heading was dewatered on August 8, 1938, disclosing an inflow of grout and muck at a maximum depth of 14 ft at the shield and running back a distance of 208 ft in the tunnel. After cleaning up, repairing equipment, and grouting the face thoroughly, progress was resumed on August 18, 1938.

Slow progress in the south tunnel (Manhattan end) was continued while the north tunnel was shut down, a total advance of 85 ft being made from June 26 to August 18. Volcanic clay and a volcanic clay-sawdust mixture were used in grouting through pipes in the heading and also through the tunnel lining to consolidate the face and cut down the escape of compressed air. On Sunday, July 17, 1938, the south tunnel was flooded to within 9 ft of the roof after a movement of the bracing at the heading resulted in rapid lowering of air pressure. From August 1 to August 10, inclusive, while the face was grouted (Fig. 11B), no advance was made in the south shield.

Tunneling conditions improved on the Manhattan side as the shields entered the permanent blanket area of the west channel. Tunnel progress from Queens, as shown on Figs. 11D and 11C, was considerably faster than from Manhattan, the material encountered being less pervious, and the air consumption much less than plant capacity; at maximum, compressed air consumption in Queens amounted to 17,250 cu ft per min of free air for the two headings (about April 5, 1939—Fig. 11C).

Excluding shutdowns and abnormally fast or slow progress, normal average progress in the shield headings, in feet per 24 hr, was:

Material	Progress
Earth	7.4
Rock	3.2
Mixed face	3.7

Based on total elapsed time, the average progress for tunneling in the river headings was 18 ft per week per heading. This over-all average includes the time for erecting shields and transferring to shield operations from rock tunneling operations, as well as all other suspensions of progress, including those due to labor disputes. (One period of suspension due to a jurisdictional dispute involving out-of-town work which had no relation to the project resulted in no progress from 8 a.m., Monday, July 3, 1939, to 8 a.m., Wednesday, July 19, 1939.)

The haulage system in the tunnels and on the surface was by storage-battery locomotives, operating on narrow gage tracks. Steel cages, shaft ways, and headframes were installed at the construction shafts, two cages, 8 ft wide by 20 ft long with a 100-cu-yd muck hopper between them, being located on each side of the river. In the tunnel heading, the mucking machine loaded directly into $2\frac{1}{2}$-cu-yd muck cars. The contractor had planned to load from the mucking machines into specially designed conveyors, which would have permitted continuous loading of the cars. However, very little use was made of the conveyors, because of union stipulations as to their operation to which the contractor would not agree and which apparently outweighed their advantages. The tunnel cars dumped directly from the cages into the hoppers; disposal was by truck.

The cast-iron lining segments for each ring were loaded on flat cars, cleaned on the surface, and delivered to the heading with the necessary grommets, washers, and bolts. The segments were lifted from the flat cars by monorail hoist behind the grouting platforms and skidded along the track rails to the erector. The lining bolts were tightened by hand. Before shoving the shield for the next ring, all bolts in the two preceding rings erected were re-tightened. The annular space between the tail of the shield and the last ring erected (a source of considerable air leakage) was calked with burlap bags.

As soon as practicable, during the erection of each ring, a 2-in. turnbuckle was placed on the back flange of the ring at the horizontal diameter. Turnbuckles were removed when observations indicated that any tendency of the ring to flatten had stopped.

The turnbuckle brackets also served as supports for the working platforms, which were towed behind the shield. Each of these platforms had two levels, one about 6 ft above the other, and, by means of the platform adjacent to the shield which was 15 ft long, access to the tunnel lining for erection, bolting, grouting, and graveling operations was obtained. The second platform, about 30 ft long, followed a few feet behind the first and carried grout pans, gravel pans, and materials, and served in connection with these operations. The monorail hoist, which was used to unload the cast-iron-lining segments, also served for carrying materials from cars to the working platforms and shield.

When the tunnel was in soft ground or mixed face, gravel was ejected from the grout holes of the lining to fill the annular space left by the tail of the shield, or the space between the rock and lining, starting at the bottom and working up on both sides of the tunnel to the three-quarter points or to top of rock when rock extended higher, except in cases where special treatment was desirable. This was followed by grouting up to top of rock, generally using a 1 : 1 mix of portland cement and sand. Both pea and $\frac{1}{2}$-in. gravel were used. In full rock sections, the void outside the lining was completely filled with gravel and grout.

Observations showed that, in general, the heavy lining in full face of soft ground tended to flatten, increasing horizontal diameters over a range of from zero to $1\frac{3}{4}$ in. (averaging about 1 in.); the opposite was true of the light lining, where horizontal diameters generally decreased after erection over a range of from 0 to 2 in. (averaging about 1 in.). This decrease occurred, as would be expected, when the gravel and grout were placed outside the lining.

Modern cast-iron lining is manufactured very nearly true to theoretical form, and practice in erection would be improved by erecting the lining as nearly to its designed shape (that is, circular) as is practically possible. This would tend to eliminate the localized strains and possible open joints which may result from distorted erection; and, with a uniformly high initial bolt stress, it would be a step in improving the waterproof qualities of the calked cast-iron tunnel ring.

Working conditions and safety precautions in compressed air and tunnel construction were in strict accordance with the best standard practice which included conformity with the Industrial Code.[5] Second tunnel bulkheads were installed in the four tunnels to provide for double-stage decompression (see Figs. 11). Three locks were generally provided in each bulkhead, the muck locks being 10.5 ft in diameter and 35 to 44 ft in length. Man locks were generally 6.5 ft in diameter and 34 ft long, and emergency locks, 5 ft in diameter and 30 ft long. Additional service locks, 5 ft in diameter and 20 ft long, were installed in each of the two first bulkheads on the Queens side.

The maximum air pressure used in tunneling was $37\frac{1}{2}$ lb per sq in. There were 745,480 decompressions with 1,145 reported cases of compressed air illness (mostly mild) with no deaths or permanent ill effects. This record may be compared with that of the Pennsylvania Railroad (East River tunnels), constructed from 1906 to 1908, in which there were 557,000 decompressions and 3,692 cases of compressed air illness, with 20 cases resulting in death.[6]

Upon removal of compressed air and completion of excavation, the tunnel lining was cleaned and calked, and concreting operations, begun on October 17, 1939, were completed on July 11, 1940. Sections of the tunnel prepared for concreting were required to remain absolutely watertight after cleaning and calking for at least one week immediately prior to placing the concrete.

Conclusions

Extended research is no longer necessary in planning and designing tunnels for highway traffic, and, with proper coordination of contracts, such projects can be completed in a practical minimum of time. The operation of existing highway tunnels will continue to provide data for improvements in design which will add to the safety, utility, and economical operation of future tunnels. Although it is possible to construct large-diameter subaqueous tunnels with safety under adverse ground conditions, a field remains for the development of improvements in construction methods which would increase tunneling progress, and reduce construction cost.

Ackowledgments

The Queens Midtown Tunnel was built, and is owned and operated as a toll facility, by the New York City Tunnel Authority, a public benefit corporation created on January 6, 1936, by act of the Legislature of the State of New York. It is authorized to construct and operate, on a self-supporting and self-liquidating basis, certain specified vehicular tunnel projects within the limits of the City of New York. The three commissioners of the Authority (Alfred B. Jones, chairman, Albert T. Johnston (died February 9, 1943), and William H. Friedman, who serve without pay, and under whose tenure of office the Queens Midtown Tunnel has been financed, constructed, and placed in operation) were originally appointed by Mayor LaGuardia in 1935 as the commissioners

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[5] *Bulletins Nos. 22 and 25*, as amended, Dept. of Labor, State of New York; effective May 1, 1922.

[6] "Compressed Air Illness," by Frederick L. Keays, *American Legislation Review*, 1912.

of the Queens Midtown Tunnel Authority, which was superseded in 1936 by the New York City Tunnel Authority. The writer served as consulting engineer to the Tunnel Authority from 1935 to April, 1936, when he was appointed chief engineer, and in that capacity directed the planning, design, and construction of the project.

During all phases of the work the Tunnel Authority has had the helpful cooperation of the municipal departments and utility companies involved, as well as of the U. S. District Engineer's Office and the representatives of the PWA. This, together with the loyalty and efficiency of the more than three-hundred members of the engineering department and the skill and cooperation of the contractors' organizations, contributed in no small measure to the completion of the project ready for operation ahead of schedule.

For the Tunnel Authority, W. McK. Griffin, M. Am. Soc. C. E., is deputy chief engineer; James H. Dugan, M. Am. Soc. C. E., engineer of design; Jacob Mechanic, Assoc. M. Am. Soc. C. E., engineer of construction; John H. Quimby, M. Am. Soc. C. E., assistant engineer of design; Leo Geenens, electrical engineer; Herbert G. Cruthers, mechanical engineer; Erling Owre, architect; John J. Nanry, M. Am. Soc. C. E., and Denzil W. Coe, M. Am. Soc. C. E., resident engineers; George L. Lucas, M. Am. Soc. C. E., materials inspection engineer; Norman D. Richardson, M. Am. Soc. C. E., engineer of contracts; David G. Baillie, Jr., Assoc. M. Am. Soc. C. E., assistant to chief engineer; Louis C. White, assistant engineer of construction. Principal consultants were Jesse B. Snow, M. Am. Soc. C. E., consulting engineer; Edward Levy, M.D., medical consultant; and Charles P. Berkey, Hon. M. Am. Soc. C. E., consulting geologist. The J. G. White Engineering Corporation and Gibbs and Hill, Inc., were electrical consultants.

In the preparation of this paper the writer has had the cooperation of many of the key members of his engineering staff, and he is particularly indebted to Mr. Baillie for his valuable assistance.

This paper has been condensed from a more extended one which has been filed at the Engineering Societies Library[6a] in New York.

[6a] 29 West 39th St., New York, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
REPORTS

WATER SUPPLY ENGINEERING

REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION FOR THE THREE YEARS ENDING DECEMBER 31, 1942

General

The preceding report of the Committee on Water Supply Engineering was for the ten-year period ending with the year 1939.[1] Previous reports of the Committee were made for the years 1934, 1935, 1936, 1937, and 1938 and were published in *Civil Engineering* or *Proceedings.*

The present report is for the three-year period ending December 31, 1942. It was undertaken with misgivings, both because of lack of time of the members who are all unusually busy with work relating to, or occasioned by, the war; and because of their inability to investigate or to speak freely regarding matters which would be of the greatest interest. Much of the report will seem trite since most readers will know so much more of particular subjects than can be encompassed in a brief report covering so many subjects. It is thought that the report will make a brief record of the state of water supply at the end of the first year of the country's participation in the greatest war of all time.

The past three years (1940, 1941, and 1942) have witnessed a gradual tapering off of government subsidized construction for civilian purposes and the substitution of expenditure for war purposes. These war expenditures began strongly in 1941 before the entrance of the United States into war in December. Examples of such activity were the advanced bases in Iceland, Greenland, Canada, Bermuda, and South America, each of which required construction of additional water supply facilities.

Since America's formal entrance into the war in December, 1941, construction of water supplies for army and navy camps, air bases, and factories has gone on furiously; and hundreds of public and privately owned supplies have had to be enlarged to carry the increased demand created by expansion of existing factories, new factories, and additional housing for the factory employees. Every maker of water supply material has been swamped with orders, and practically all of them have expanded their operations to include other materials. Cast-iron pipe foundries, for example, have begun making steel castings, shell casings, and similar items for various war purposes. At the

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Note.—Please forward all comments on this Report directly to Chairman Thomas H. Wiggin, 90 Broad St., New York, N. Y.

[1] *Transactions*, Am. Soc. C. E., Vol. 105 (1940), p. 1740.

same time, manufacture along regular lines, such as cast-iron pipe and fittings, has been pursued to the point, as one foundryman put it, of finding out for the first time just what his plant was capable of producing. In Table 1 are given approximate statistics furnished by several cast-iron pipe foundries on comparative production in 1940, 1941, and 1942. In some cases these figures are related to production for the previous years—1937, 1938, and 1939.

TABLE 1.—DATA ON PRODUCTION OF CAST-IRON PIPE AND CEMENT LINING

Year	Foundry A (%)	Foundry B		Foundry C		Foundry D (%)
		Tons	%	Tons	%	
			(*a*) PRODUCTION			
1937–38–39	....	....	....	700,000	100	100
1940	100	82,700	100	800,000	114	95
1941	130	140,800	170	1,100,000	157	152
1942	140	156,500	190	1,200,000	172	163
			(*b*) STANDARD CEMENT LINING			
1937–38–39	....[a]	....	....[b]	....	3[b]	11.5[c]
1940	100	....	3.76[b]	....	4[b]	12[c]
1941	105	....	4.22[b]	....	1.5[b]	6[c]
1942	60	....	3.69[b]	....	3[b]	2[c]
			(*c*) THIN CEMENT LINING			
1937–38–39	....[d]	....	....	....	37	....
1940	....[d]	....	8.86	....	30	....
1941	....[d]	....	8.32	....	20	1
1942	....[d]	....	2.64	....	10	0.5

[a] Proportionately more than in 1940. [b] Probably on tonnage, rather than footage basis. [c] On tonnage basis. [d] Thin and standard linings are lumped under "Standard."

It will be seen that consumption went on rapidly in 1940, 1941, and 1942, being 40% to 90% greater in 1942 than in 1939 or 1940. The statistics include also percentages of cement lining both of standard thickness and of the thin variety often called "Enameline." Cement lining held its own fairly well in total tonnage but generally fell off in percentage. The Committee knows of one case where inability to get labor crippled the cement lining department. Some information indicates that federal government agencies were less likely to specify cement lining, so that this department of the business was apparently not expanded to keep pace on a percentage basis with the general expansion of production.

In January, 1943, there appeared to be indications of a reduction in the demand for water pipe.

Steel Pipe.—One manufacturer of steel pipe, writing from a general knowledge of the business, informed the Committee that the quantities of steel pipe made for the water works of cities and towns and private utilities have been less than usual but the water pipe supplied to ordnance plants, cantonments, and industrial plants combined with that for cities, towns, and private utilities has made the total production of steel water pipe an all-time high. In addition, there was a huge demand for steel pipe for other uses, making the grand total also an all-time maximum.

Reinforced Concrete Pipe.—Production of reinforced concrete pipe during the past year has been far in excess of former years. Again the main quantity has gone to war plants, but considerable quantities have gone to the enlargement of municipal supplies serving war plants, shipping, and similar needs. A prestressed reinforced concrete pipe has been developed which economizes material. Its only use so far has been for oil supply lines to the Navy. Such a development has been under study for some years, and its value has been anticipated by engineers who have experimented with reinforced concrete pipe or have had pre-load tanks.

Cement-Asbestos Pipe.—Manufacturers of cement-asbestos pipe have been working at top output during 1942. The fact that little or no iron is needed for this type of pipe induced the Office of Production Management (O.P.M.) and to a less extent its successor, the War Production Board (W.P.B.), to order its use somewhat indiscriminately instead of cast iron or steel; but such substitutions in the experience of the Committee could not be made because the cement-asbestos product was then already oversold. This was due in part to an early recognition that this type of pipe took less ship tonnage per foot, so that it was specified for overseas use.

A new type of cement-asbestos pipe has been manufactured, not by winding successive thin layers over mandrels, as in the older pipe, but by extruding a fairly stiff mortar containing asbestos. The Committee is not informed whether this pipe is being supplied commercially.

Lining of Large Pipe in Place.—The process of lining large pipe in place, using a traveling centrifugal mortar-applying machine with revolving troweling attachment, has continued in the field. A new steel pipe at Toledo, Ohio, was so lined. The flow coefficients of this type of pipe have been shown to equal those of new pipe of smooth construction.

Over 40 miles of pipe in sizes 36 to 93 in. have been so lined, and work is in progress on contracts aggregating about 16 miles. Among the water supplies for which this process has been used are those of Newark and Trenton, N. J.; Toledo, Cincinnati, and Akron, Ohio; the Boston, Mass., Metropolitan District; Grand Rapids, Mich.; and Bethlehem, Pa.

Lining of Small Pipe in Place.—The American branch office promoting the Australian process of applying cement lining to small pipe in place has continued to do considerable business. Among the places where one or more contracts have been executed or are in process are West Palm Beach, Fla.; Shamokin, Mauch Chunk, and Indiana, Pa.; Charleston, S. C.; Falmouth, Newton, Danvers, and Milton, Mass.; New Albany, Ind.; Fort Hancock, N. J.; and El Centro and San Diego, Calif. A total of about fifty miles of pipe has been so lined. Tuberculated pipes having low coefficients of flow are brought up to coefficients practically equal to those of new pipe reckoned on the actual diameter of the lining. Smaller pipe, such as 6 in., and in greater measure 4 in., suffers some reduction in coefficient when reckoned on the nominal size, owing to the fact that the lining cuts about 0.5 in. from the diameter. The coefficients reckoned on the nominal diameter, however, are generally 115 or more, and the capacity tests of the lined pipe generally show double or more the capacity of the tuberculated pipe.

PRIORITIES

For the record, and for a proper understanding of the water supply engineering problems of the latter part of the period covered by this report, it is necessary to include some statement of the control of water works supplies by the federal government. This is a part of the control of all supplies on account of the need for conserving every possible pound of metal or other critical material for war purposes.

The salient features of this control are as follows:

1. All projects require submission of applications to the W.P.B. on forms provided, giving war-need justification for the project; lists and cost of material and labor required; weights of cast iron, steel, copper, brass, and lead involved; and other pertinent information. Such applications are examined in Washington, D. C., by appropriate groups of the W.P.B. If approved by the first group, these applications go to committees of general control called, for brevity, "Iron and Steel," "Brass and Copper," and so forth; and if still approved go to "Issuance," a group from which certificates of priority are issued giving ratings indicating the relative importance of the project as compared with ships, guns, cantonments, airplanes, tanks, new manufacturing plants, and other of the myriad works which are in the mad scramble for the insufficient material available.
2. Materials for maintaining utilities and for small extensions to serve houses whose foundations were completed before a certain time in districts which in general are already served may be obtained or used if available in stock; but limits have been placed on the total of such extensions, such limits having relation to the past history of the plant.
3. Authorization to obtain material to repair breakdowns in pumps and other equipment can be obtained by telephone or telegraphic communication with the W.P.B. and the Board's telegraphic authorization to the company which can supply the material. Confirming papers are required.
4. Material for water supply connections to authorized manufacturing plants, generally government owned, to authorized housing developments, or to other war projects can be obtained by use of the priorities which attach to such war projects.
5. Inventories of water supply material on hand have to be filed periodically with the W.P.B. which has further directed that inventories be reduced to 60% of those of December, 1940. A plant may call on another to supply surplus material at cost prices providing the project has a rating of A-1-f or better. This interchange of material has not progressed very far as yet. Municipalities and private companies have found great difficulty in complying with the inventory reduction order.
6. The W.P.B. may, and often does, change the kind of material requested to some other kind less needed at the time for war purposes. Thus, wooden

pipe, cement-asbestos pipe, or concrete pipe may be substituted by the W.P.B. for cast iron or steel; copper service pipe has been eliminated and steel or lead substituted; iron glands and rings have been substituted for bronze in pumps; steel for bronze in hydrant stems; wood for steel in tanks; ground storage reservoirs for elevated tanks; and substitute compounds for jute have been used in pipe joints. Some of these substitutions require reconsideration at times because of lack of the substitute.

The application of the priority system has been attended, of course, with delay both in action by the W.P.B. in assigning priorities and in the supply of material. A typical example of a project for facilities to serve new war plants and housing is as follows:

(*a*) Application made December 29, 1941.
(*b*) Priority rating received February 9, 1942—amount of improvement approximately $200,000.
(*c*) Priority ratings granted, A-2 and A-6. At that time ratings ran in descending scale through A-1-a, A-1-b, etc., to A-1-f; then A-2, A-3, A-4, and so forth.
(*d*) Installation needed by June 15, 1942. It included about 14,000 ft of 20-in. and 24-in. cast-iron pipe and five electrically driven pumps. Most of the pipe was received and laid by June 15, also three of the pumps and one motor. Then there began an avalanche of war orders for motors; the A-2 and A-6 priorities became worthless; the other pumps and motors could not be delivered because of the insertion of a lot of AA-1, AA-2, and similar priorities at the head of the list where formerly A-1-a was the top (except an AA for emergency Army and Navy work).
(*e*) A succession of telegrams, telephone calls, letters, and visits to the W.P.B. at Washington secured increasingly more favorable ratings on the undelivered material, such as A-1-c, AA-3, and AA-2. The last of the five motors was shipped late in 1942, and the whole job will probably be completed by April, 1943, instead of June, 1942.

Such examples could be multiplied out of personal experience of the Committee. No criticism is intended by these remarks. There was simply too much demand and too limited a supply. Some of the delay was due to an insufficient sanitary engineering personnel in the O.P.M. and its successor, the W.P.B., and some to the supervisory committees like "Iron and Steel" and "Brass and Copper" in the evaluation of all kinds of projects in deciding which shall share in the limited supply of material. More prompt action, right or wrong, would have saved much engineering effort and expense, but could not have cured the scarcity of materials. In general, the delays probably have aided in the production of war goods. The 1942 weather in the East was so favorable as to well spaced and liberal rainfall that delays to water supply installations did not actually cause the shortage that was feared. A summer like that in 1941 would have caused shortage.

The handling of applications by the W.P.B. is becoming more prompt and skilful; also, the number of civilian projects is diminishing and new methods of material allotment are in view.

NOTABLE CIVILIAN WATER SUPPLY PROJECTS

In this brief summary of activities the Committee deems that note should be made of a few large civilian water supply projects.

Grand Coulee Dam.—The Grand Coulee Dam (State of Washington) was first filled to spillway level on June 1, 1942. This reservoir is 151 miles long. The spillway is 1,650 ft long and 300 ft high.

Lake Mead, Boulder Dam (Arizona-Nevada).—Lake Mead, 120 miles long and containing 32,360,000 acre-ft, or 10,400 billion gal, first spilled over on August 6, 1941.

Central Valley Project.—This project, in California, which embraces the huge Shasta Dam, also Keswick Dam, Friant reservoir, and a system of canals, was proceeding toward construction at the great speed for which these huge western projects are noted when work was stopped by the W.P.B. The masonry of Friant Dam has been completed but outlet works and canals have not been built, so that no use can be made of the work. The gates of Friant Dam were closed on October 20, 1941, and storage of water was begun. This Central Valley project is for hydroelectric power, irrigation, flood control, and water supply, the power being developed at Shasta and Keswick dams. To those who have lived or traveled in California and the dryer parts of the West, the significance of this huge project, particularly in irrigation, will be clear. John C. Page, M. Am. Soc. C. E., commissioner of the Bureau of Reclamation, has stated that the economic foundation of the West is in the utilization of its water supply for all purposes. In all the West 20,000,000 acres are now being wholly or partly irrigated, and there is water enough to completely irrigate 40,000,000 acres after full development.

Metropolitan Water District of Southern California.—This project includes Parker Dam on the Colorado River, 242 miles of aqueduct of many types from Parker Dam to the vicinity of Los Angeles, Calif., several large booster pumping stations at the dam and along the aqueduct line, a terminal reservoir of 35,000 million gal capacity, a filter and softening plant of 100-mgd capacity planned for an ultimate capacity of 400 mgd, and various distribution pipe lines. The aqueduct has an ultimate capacity of 1,605 mgd, about 10% of which is said to be used by present consumers.

The initial installation project was completed in 1941 and put into service. The cost was $23,000,000 less than the authorized bond issue of $220,000,000.

Mono Basin Supply.—In the fall of 1941, the Los Angeles Bureau of Water Works and Supply celebrated the completion of the extension of the Owens River watershed into the Mono Basin. This required seven years, the work being done by the city. It involved an 11-mile tunnel, driven through the Mono Craters Mountains, together with a dam in the Owens Valley, some 300 miles north of Los Angeles, to store 60,000 million gal. The new watershed is expected to deliver 365 cu ft per sec. The cost of the work was about $20,000,000.

Delaware-Rondout Water Supply for New York, N. Y.—This project, estimated to cost about $280,000,000, is for an additional supply of about 540 mgd from Rondout Creek and the East Branch of the Delaware River westerly from the present Catskill supply. Of the 85-mile Delaware aqueduct, all a deep pressure tunnel in rock, the southerly 14 miles from Kensico reservoir to the Hill View distributing reservoir at the northerly limit of the city is finished and ready for emergency use. The middle section, from West Branch reservoir to Kensico reservoir, 26 miles, including the by-passes under both reservoirs, is finished and soon will be ready for emergency use. The permanent operating equipment for the gate-houses at these two reservoirs being unobtainable under wartime restrictions, control devices are being improvised. The 45-mile northerly section from Rondout reservoir to West Branch reservoir is finished except for one-half mile of concrete lining, some grouting, the closure of four shafts, and the gate-house at the West Branch end. It should be completed, except for permanent operating equipment, in the summer of 1943.

Of the three impounding reservoirs, that on the East Branch of the Delaware is the most distant and the one to be last constructed. Here surveys and the comprehensive boring of the dam site are under way. At the Neversink reservoir, the second to be built, the stream control works (including a 30-ft diameter concrete lined diversion tunnel with inlet channel and a masonry lined outlet channel one-quarter mile long) and the cofferdam are substantially completed; and closure of the cofferdam is under way.

At the Rondout reservoir, that nearest to the city, the cutoff wall, which is of the caisson type, and the relocated highways have been completed; but work on the rolled embankment of the Merriman Dam and on the aqueduct gate-house have been substantially suspended since 1941 due to the commandeering of the contractor's earthmoving equipment and the unavailability of the necessary metal work. As an emergency measure, a small tunnel to deliver run-of-river supply up to about 400 mgd is being driven from the outlet channel of the diversion tunnel to shaft 1 of the Delaware aqueduct. Progress on all structures should be such that, with improvised control devices, some water can be delivered from Rondout Creek to the city in the summer of 1943.

Metropolitan District of Boston.—During 1941, the new supply of about 210 mgd from the Ware and Swift rivers came into full use, thus adding nearly double as much to the 120-mgd capacity of the Wachusett and North Sudbury systems which the new supply supplements. A plan is now under consideration for a deep tunnel to supplement surface aqueducts and pipes in a portion of the transmission system ending at Chestnut Hill reservoir. This is designed to increase the safety of the supply in case of enemy attacks.

Baltimore (Md.) Water Tunnel.—Baltimore has added a 7-mile pressure tunnel to deliver the water supply from the Loch Raven Dam to the Montebello filters. This tunnel, of 12-ft finished diameter, is concrete lined. It has a capacity of 275 mgd, with a 10-ft loss of head. The cost was about $5,400,000. Two thirds of the tunnel is lined with concrete, with about 12,000 ft lined with a continuous welded steel plate cylinder, with concrete backing and gunite mortar lining. This tunnel was begun in 1937 and completed at the end of 1940. Previously, Baltimore had one unlined 12-ft diameter tunnel to deliver the water supply of the city.

Indianapolis, Ind.—Indianapolis fortunately began construction on a new 7 billion gal impounding reservoir, a first or 16-mgd unit of filter plant, the revamping of its Fall Creek pumping station, and a large mileage of main extensions before priorities began to interfere with normal procedure. The water demand in 1941 was about 37 mgd, which was 12% higher than that in 1940. The first six months of 1942 showed a further increase of 11%.

Toledo, Ohio.—The pollution and insecurity of the Toledo water supply, obtained from the Maumee River, caused the city to construct a new system, taking water from Lake Erie, some 2½ miles offshore. Water is carried in a pipe line from the crib to the shore, through 108-in. concrete pipe. A low service pumping station delivers the water through some 9 miles of pipe to an 80-mgd treatment plant. A new high service pumping station pumps the water through 2 miles of 72-in. pipe to a tunnel constructed under the Maumee River. This work cost about $10,000,000, and was practically completed in 1941.

Harrisburg, Pa.—This city, which since 1905 has purified Susquehanna River water as its supply in one of the pioneer masonry rapid sand filter plants, has now constructed an upland impounded supply which at present, so far as the Committee is informed, is not filtered.

WATER SUPPLY PROJECTS DIRECTLY RELATED TO WAR

Cantonments and Other Military Posts.—Numerous cantonments and military posts have been built at scattered points throughout the world and old ones have been enlarged. These have all required new or extended water supplies. The War Department has kindly supplied the Committee with some statistics from which the following data have been taken:

Stations using purchased water	334
Stations having some type of water supply plant	710
Stations having filtration and treatment plants	91
Filtration and treatment plants	144
Estimated water usage at all stations, as a minimum (mgd)	500
Total capacity of filter plants (mgd)	191
Miles of water main, maximum diameter 30 in	6,608
Miles of service pipe	2,680
Civilian personnel assigned to maintenance and operation of water supply systems (no enlisted men employed), approximately	3,000

The mileage of water main is greater than that of New York City and the water consumption is well over half that of New York City.

The daily per capita allowance is about 100 gal as compared with about 50 gal per capita in the United States and 1.5 to 25 gal per capita for the expeditionary forces in the World War ending in 1918. Direct sanitary flushing fixtures instead of flushing fixtures of the tank type are used in the present equipment. This condition and the simultaneous habits of military organizations have combined to make the peak demands very heavy, so that the allowance of 100 gal per capita has not been found to be too large.

Detroit. Mich.—The tremendous expansion of war plants in Detroit has required many new supply lines. Some of these are larger pipe replacing sizes of 16 in. or 20 in. in crowded streets. The total length of new main financed by the Federal Works Agency (FWA) is stated to be about 60 miles, costing about $3,600,000.

Mill Creek Supply.—This 15-mgd supply for the Wright Aeronautical Corporation at Lockland, Ohio, includes eleven gravel packed wells at the Miami field south of Hamilton, Ohio, twenty miles of 36-in. and 42-in. concrete transmission main, and a 15-mgd reservoir. The pipe follows the hydraulic gradient as nearly as practicable. Only 36 lb of steel per linear foot of 36-in. concrete pipe were used. This is the weight of a steel shell 36 in. in diameter and 0.093 in. thick, or about one third the thickness of a thin steel pipe which might be used. The reservoir was in balanced cut and fill with plain concrete lining, and roof of timber construction. This $3,000,000 project required less than 1,500 tons of critical material.

Houston, Tex.—A raw water supply consisting of a low-lift pumping station on the San Jacinto River and a canal system to carry the water ten to twenty miles to the industries located along the Houston Ship Channel is a FWA project in Houston. This supply supplements the overworked ground supply.

Newport News, Norfolk, and Portsmouth, Va.—New water works costing about $10,400,000 have been built or are in process of construction for these seaport cities in the Hampton Roads area. They are needed largely because of the enormous increase in shipbuilding and in naval and military activities. More than half of these works are government owned and the remainder were government financed with municipal participation of $1,500,000.

The new works include intakes on the Chickahominy and Nottaway rivers; 81½ miles of reinforced concrete pipe of various sizes from 42 in. down to 24 in.; 17 miles of cast-iron transmission main 16 in., 20 in., and 30 in. in diameter; two 25-mgd pumping stations on the Blackwater and the Nottaway rivers, respectively; two 6-mgd filter plants and one 12-mgd addition to a filter plant; two reservoirs of 2.0 and 3.76 billion gal, respectively; and other appurtenant structures.

San Diego, Calif.—This city is said to have had the greatest proportionate growth in population of all cities in the United States as a result of war activities. Its population of 202,000 in 1940 has become 350,000. Water supply and other facilities have had to be enlarged to meet this suddenly increased demand and plans that had been visualized for about the year 1960 have had to be materialized at once. The 26.6 mgd estimated safe yield of its five storage reservoirs is sufficient only for a population of about 250,000—hence another storage reservoir is being constructed on a tributary of the San Diego River below the existing El Capitan Dam. The new reservoir will cost about $3,550,000 and will add about 5 mgd to the safe yield of the system. Work is being rushed with the expectation of completion in February, 1943, in time to impound what may be left of the winter runoff. This is a dry country and the cycle of fill and draw for storage reservoirs extends not simply over a year or two but over perhaps eight or ten years. Transmission and distribution mains costing nearly $3,500,000 are also being installed, with the aid of about $2,100,000 in federal funds. The new dam, called San Vincente Dam, is a straight gravity structure, 950 ft long and 200 ft high, with a 275-ft spillway in the center. It is constructed of concrete. The reservoir has a capacity of 75,000 acre-ft, or about 25 billion gal.

Gibraltar, Spain.—The supply for this fortress was formerly from sixty acres of catchment areas partly surfaced with concrete. Since 1936, two wells have furnished most of the drinking water. Reservoirs of an aggregate capacity of 24 million gal under 60 ft of rock cover have now been added, also a water distilling plant.

COORDINATION AND DEFENSE OF WATER SUPPLIES IN WARTIME

A telegram dated December 8, 1941, the day after the Japanese surprise attack on Pearl Harbor, sent by the U. S. surgeon general to state departments of health, urged water works officials:

(1) To take immediate steps against sabotage;
(2) To provide guards at danger points; and
(3) To step up chlorine dosage to provide a higher residual throughout the distribution system. This last request has not met with general compliance on account of the disagreeable effect on the quality of the water and the feeling that it is not necessary.

Much study based on English experience had been given to water supply problems in war previous to this time by water works associations and organizations.

Prevention of Sabotage.—Guards, fencing, and flood lighting are being used by important water supply plants to the extent of their financial ability. Except in a few critical areas where the Army has occupied the region of water works property, and except temporarily for a short period after the surprise attack of the Japanese at Pearl Harbor on December 7, 1941, the protection of water works property has been laid by the Army and Office of Civilian Defense (O.C.D.) at the door of the utilities. The pleas of the utilities, whether municipal or private, for military guards at public expense generally have gone unheeded, though state guards have been furnished in some instances for part of the force, and police have functioned in the city areas.

Considerable coercion has been exercised by the Army, and in some cases by the state, in compelling such protection. In New York State, for example, penalties have been made applicable for non-compliance. Large additions in operating expense have been incurred because of the employment of three shifts of guards at important points. So far as the Committee knows, no increase in water rates has been permitted on account of these expenses. Such increases for private utilities must be approved by public service commissions, who regard the increased business as an offset to the cost of guarding without considering that the increased costs of labor and taxes are more than sufficient in general to offset the gains from additional business. The Office of Price Administration also has general jurisdiction over water rates as a part of its attempt to prevent higher prices. The Committee has been informed that there has been a relaxation in the guarding in several areas, and perhaps this is general.

Fencing and flood lighting at dams, pumping stations, and filter plants have been adopted commonly. Wood has had to be used for some of the fences on account of the scarcity of steel and the consequent inability to obtain priorities on steel.

There is a considerable difference of opinion even among experts as to the effectiveness of guards and lighting. By some it is thought that organized sabotage would evade or overpower by surprise attack any guards that could be afforded, though the rare cases of individual "crackpot" vandalism generally would be prevented. The systematic work of the Federal Bureau of Investigation (F.B.I.) in investigating aliens and enemy sympathizers throughout the country and particularly among employees of key plants, and in causing arrest or internment of dangerous persons, is thought by many to be the main safeguard because it largely prevents organized sabotage. Fingerprinting of water works employees has become a rather general practice, and these fingerprints are handled through local police for final examination by the F.B.I.

In water supplies having many outlying storage reservoirs, general patrolling is impracticable and various devices have been used, such as periodic car patrol or boat patrol with telephonic control, either with or without benefit of two-way radio telephones. Fishing and boating privileges have been generally rescinded, at least for areas of water near dams. The habitual fishermen demur at this, saying that they are the best guards obtainable and that they work without wages.

Contamination by Poison or Disease Germs.—This form of sabotage has been given much thought and discussion. Some operating organizations, notably Los Angeles and Detroit, have greatly increased the number and scope of their laboratory examinations, even to hourly analysis of samples from key points in their systems. Chlorine is a destroyer of most poisons as well as germs. Its use in greater quantities has been urged by certain public health officials. The maintenance of a 0.4-ppm residual of chlorine in all parts of the system was the recommendation. Much opposition to this policy exists and, so far as this Committee is informed, the practice has not been generally adopted in municipal or private water companies.

The consensus among sanitary experts seems to be that it takes so great a quantity of poison to cause damage to health, or death, and the tastes are generally so noticeable that the danger of poisoning a large water supply is small. Disease germs are likewise difficult to inject on a large scale and are killed by chlorine. Small-scale application of disease germs is a recognized possibility, but no cases of this kind of sabotage have been reported. In territory taken over by advancing troops, poisoning and contamination of water supply abandoned by retreating forces are common, and no advancing troops omit thorough precautions before use of such supplies.

Decontamination.—In cooperation with the Massachusetts Committee on Public Safety, the O.C.D., and the U. S. Army, the Division of Sanitary Engineering has formulated a program of decontamination of areas polluted by war gas, including sources of water supplies and areas where foods are stored. In connection with this program, four schools were conducted at the Massachusetts State College, at Amherst; 181 chemists and bacteriologists have been trained at these schools and are now available to identify war gases and detect pollution of water mains following enemy bombing.

Precautions in Preparation for Bombing Attacks.—Water works organizations of all kinds throughout the country have been working on preparations for bombing attacks since the Pearl Harbor attack on December 7, 1941. Many had studied the English experience months earlier, both as a matter of interest and because expectation of war was general before Pearl Harbor. Citizen committees of all kinds were formed to study procedure for eliminating, during threatened air raids, and obscuring, at other times, all street and all building lighting visible from without; for giving first-aid to the injured; for providing and marking the safer points of refuge; for extinguishing individual incendiary bombs and conflagrations; for shutting off broken water mains; for providing safe drinking water to areas where water supplies have failed due to bombing; and for other functions found by English experience to be necessary in case of bombing. These citizen activities finally became focused in the O.C.D., first headed by the Hon. Fiorello LaGuardia, Mayor of New York City, and Mrs. Eleanor Roosevelt, wife of the President of the United States.

In the meantime, the myriad water supply associations, including preeminently the country-wide American Water Works Association and the New England Water Works Association, also the state boards of health, were not behind. Through many local and general meetings and committees they developed the technique of safeguarding water supplies and restoring them to service as quickly as possible when damaged.

This report could be made of book-size, devoted to this subject alone, and still leave untouched many of the available data. For the record a mere catalog of salient principles developed with the aid of English experience will be given. These principles, as they have been sorted out by the Committee, are as follows:

1. The mapping or the improvement of existing maps of the systems, with particular reference to location of valves to permit shutting of broken mains and preventing unnecessary drainage of the system; in many cases the making of special valve location sketches with ties.

2. The inspection of valves to see that they can be operated and will be reasonably tight.

3. The installation of additional valves to reduce the size of shut-off areas. (This practice has been emphasized by the British—but probably has not been followed to any large extent in the United States.)

4. The making of cross connections between adjacent water supply systems wherever practicable—this is also a lesson from the British. A great many cross connections have been made. One difficulty is the small size of those outlying pipes which are near enough to be connected at reasonable expense. In important cases federal aid has been sought, but it is very slow in materializing after initial approval. Initial legal inability of municipal plants to go outside their corporate limits has been remedied by legislation in some states.

5. Providing supplies of quick repair equipment for broken mains. Such joints have been designed in imitation of and improvement over English devices, but this Committee believes that scarcity of funds and of material allotted by the W.P.B. has prevented any considerable stocking of such material. The

same has become true also for common pipe line supplies. At first, considerable stocking was done, but this was forbidden by the W.P.B.; and what they consider as over-stocks are being used or are to be used for approved extensions, one utility being permitted to buy from the over-stock of another.

6. The provision of means for applying chlorine or hypochlorite of lime solution in very liberal quantities in stretches of pipe near breaks. English experience has shown that contamination from sewers or sewage-polluted earth may be driven far into the pipe at such points by bomb explosions. Considerable quantities of such sterilizing equipment have been stocked by water utilities, but the companies supplying such equipment no longer can supply much on account of demand by the Army and the government for lending or leasing to allied nations.

7. The storage of chlorine and chlorine equipment at central points in some states.

8. Interchange of men and equipment between bombed areas.

9. Last and most important, organization for mutual aid, repair, and management during and after raids. The general pattern has been a state organization of civilian defense under the general supervision of the O.C.D. with districts within the state and municipal subdistricts. A water coordinator for the state together with branch coordinators in districts is given general supervision of water works operation in case of bombing. They, in turn, must fit themselves into the general picture of state civilian defense administration. Sometimes the state governor assumes the headship; sometimes an appointee of the governor.

Hundreds of courses have been given and have been attended by water coordinators and utility personnel. Additional personnel have been enlisted for voluntary aid in doing the increased work entailed by bombing, and, although all parts of the country seem keyed for a crisis in bombing, fortunately their training has not yet required use. Military experts hold various opinions as to the likelihood of bombing, but most have said that they expected some bombing from expeditions based in Canada or from long-range bombers or other sources; and they have not limited the danger zones to any part of the country. The new geography of great circles available to airplanes has shown some remarkable changes in accessibility.

Camouflage of Water Supply Works.—The Committee knows of elaborate studies which have been made for camouflaging one large and critically located dam. A cheap camouflage material has been found which will float and can be removed easily and cheaply and replaced in different colors to follow the seasons. The Committee has been informed that there has been an effort to camouflage some tanks and reservoirs on the West Coast.

Protection of Dams.—There has been considerable discussion at meetings of water supply groups and committees as to the vulnerability of large gravity dams to bombing attacks. The Committee is not aware that any report on this subject has been made available to water supply officials by military or other experts from whom information has been sought; but it has heard informal views expressed by civilian engineers who have relations with the military authorities. At least some of these authorities appear to believe that demolition of large dams by aerial bombs is possible but that such dams are just another hazard, not more important than a big war plant, and must take their chances with the other important facilities for supporting the war effort. This view is apparently colored by the present trend of the war which, except for hostile submarines, is away from our shores. The Committee regards protection of large dams as a proper subject for inquiry by owners and defense officials having responsibility for such structures, particularly those in critical locations where demolition would cause large loss of life or would cripple vital water supplies. In a war which is likely to cost the United States several hundred billion dollars, the expenditure of a few millions to protect vital water supply structures would not seem out of proportion. The Committee feels that the relative magnitude of the catastrophe which would be caused by loss of the principal storage reservoir of a large city is not fully understood by those who are not regularly connected with water supply operation. A loss of all stored water, apart from loss of life, might not be replaceable for a year or even several years.

Design of Dams.—In former reports the Committee has noted the lack of authoritative pronunciamentos from this Society regarding proper provisions for uplift water pressure and ice pressure in designing new dams and checking old ones. Committees have been appointed but have not functioned. These questions have not lost their importance by the Society's neglect in answering them.

Droughts, Floods, and Spillways

A severe drought occurred in the northeasterly part of the United States during 1941 and the early part of 1942. Rainfall deficiencies in 1941 were from 11 in. to 27 in. in Massachusetts and Connecticut. Some of the streams had their lowest flows of record. At the same time, in Kansas and another section of the Middle West, extremely heavy rainfall caused stream flows that in the Kansas area reached nearly a hundred times the normal October flows and were greater than any recorded previously. Storage in the reservoirs of the City of New York was reduced almost as much as in the record drought of 1930–1931, and the skilful publicity campaign to conserve water had to be augmented. Due in part to this campaign and in part to migration of population with the war, New York City now has a water consumption as low as in the year of most severe depression, 1932.

The City of Rochester, N. Y., also had very low storage. It had to purchase water from other sources.

Floods.—Various parts of the country have experienced floods of record intensity during the past three years. At the end of February, 1940, rainfall of 8 in. or more on the watershed of the Sacramento River in California created its greatest recorded flood. At Kennett, Calif., the river rose 3 ft above previous records. Had the Shasta Dam been available for storage, this flood would have been checked.

In southern New Jersey, on September 1, 1940, there was a rainfall of approximately 10 in. The runoff from this rain caused the progressive failure of a number of small dams, and several municipal water supplies were temporarily put out of service. This flood caused the failure of small structures that had withstood floods since Colonial Days.

Between May 17 and 23, 1942, rainfall aggregating up to 7.85 in. occurred in the watersheds of the Susquehanna, Lehigh-Schuylkill, and Lackawanna rivers in Pennsylvania. Much of this rain fell in the early evening of May 22 and great damage was done. One small masonry dam that had stood for about seventy years was completely breached, and many others were somewhat overtopped and damaged. The computed runoff from several small watersheds of 2 to 4 sq miles was about 400 cu ft per sec per sq mile. The region thereabout is in part glacial moraine and the stream beds were torn most violently. Gravel and large boulders were carried downstream, blocking regular channels in many places. The streams there moved delta-wise into new channels, washing out bridges and concrete highways together with long sections aggregating thousands of feet of water mains, some as large as 30 in.

The storm of May, 1942, however, was completely eclipsed by one of July 18–20, 1942, in the region of Port Allegany, Pa., partly in the Susquehanna and partly in the Ohio river basins. Rainfalls of tropical intensity occurred. An intensive study of all rainfall data of any kind, including water pail, wheelbarrow, and other odd measurements, is being made by the U. S. Weather Bureau. Reports showed rainfalls up to 38 in., and later reports indicate that 35 in. was certainly reached, mostly in 18 hr and much of it in 12 hr. J. W. Mangan, M. Am. Soc. C. E., district engineer of Pennsylvania State Hydrographic Service, permits the Committee to quote him: "There is no doubt that, within an area of at least 500 square miles, the precipitation was the greatest by far ever recorded for 12 to 18 hour storm in Pennsylvania and has exceeded that recorded previously in northeastern United States."

No member of the Committee visited this area after the storm, but eyewitnesses report the damage as horrendous. Many flood flow runoff estimates have been made for this storm. Among those to be published by the State of Pennsylvania are the following: 2,100 cu ft per sec per sq mile for 11.4 sq miles; 2,300 cu ft per sec per sq mile for 6.59 sq miles; 2,400 cu ft per sec per sq mile for 6.73 sq miles; 2,100 cu ft per sec per sq mile for 7.06 sq miles; 3,300 cu ft per sec per sq mile for 0.1 sq mile;[2] 3,600 cu ft per sec per sq mile for 0.4 sq mile;[2] and 2,400 cu ft per sec per sq mile for 2.14 sq miles.

Space limitation prevents more than passing reference to floods of 1942 in Maryland, Virginia, and West Virginia in the Potomac Valley and that of the Ohio River caused by 63 hr of rainfall ending on December 31, 1942.

Spillways.—The tendency to make spillways more liberal continues as records of large floods are added. It is not a cheerful thought that many existing dams are bound to go out with lapse of time as storms of high intensity chance to hit the valleys in which they lie. While it would be physically possible to remove the danger, it is not economically possible, just as protection of all buildings from lightning is physically but not economically possible. The degree of protection must depend on the magnitude of the disaster which would be caused by the failure.

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[2] Areas of watersheds in doubt.

There is, however, a decided tendency toward larger spillways caused, in part, by better engineering records of storms which tend to give engineers in each locality a more lively impression of the chances of greater floods in their various localities, also in part by the participation in recent years of federal authorities who are less influenced by the economic considerations that have a great part—often too great a part—in local decisions. As an indication of the upward tendency in spillway design, Table 2 is given.

TABLE 2.—FLOODS RECOMMENDED FOR CONSIDERATION AS BASIS OF SPILLWAY DESIGN

Area, A, of watershed (sq miles) (1)	SPILLWAY CAPACITY, Q, IN CUBIC FEET PER SECOND PER SQUARE MILE				
	1919 regulations of an eastern state (2)	From a 1941 curve by same state[a] (3)	U. S. Engineers formula, used considerably, $Q = \frac{5,000}{\sqrt{A}}$ (4)	From suggested maximum for eastern states in "low dams,"[b] $Q = \frac{6,000}{\sqrt{A}}$ (5)	Curve required to enclose Pennsylvania storm of July, 1942,[c] $Q = \frac{7,100}{\sqrt{A}}$ (6)
1	600	3,000	5,000	6,000	7,100
10	365	1,100	1,580	1,900	2,250
20	270	790	1,110	1,340	1,590
50	180	510	7,100	850	....
100	145	370	....	....	....
1,000	75	135	....	....	....

[a] Based on an inclusive curve of floods up to that date but since exceeded. This state has now no fixed maximum provisions for spillways but determines quantities according to local conditions of watershed, pondage, damage which destruction of dams would involve, etc. [b] Published by the National Resources Committee in 1938. [c] In the region of Port Allegany. The maximum case, namely 2,100 cu ft per sec per sq mile on 11.4 sq miles, would require the formula $Q = \frac{7,100}{\sqrt{A}}$. Others in order would require numerators of 6,250, 5,900, and 5,600, respectively.

In the July 1, 1942, issue of the *Delaware Water Supply News*, Francis B. Marsh, M. Am. Soc. C. E., designing engineer of the New York City Board of Water Supply, states that from a study of flood flows and the formulas devised for estimating the maximum flood flows he has concluded that the best modern formula is the one used for the design of the Hiwassee Dam, where the drainage area is 977 sq miles. The formula is $R = \frac{5,000}{\sqrt{A}}$, in which R is the maximum rate of runoff in cubic feet per second per square mile and A is the area in square miles. Mr. Marsh states that in the small areas, between 10 and 100 sq miles, this formula might be modified to read $R = \frac{6,000 - 10\,A}{\sqrt{A}}$.

It is surprising how small some spillways have been made and are still being made where there is no governmental compulsion and even where there is supposed to be governmental supervision. Many, perhaps most, dams 40 to 75 years or more in age are found to have spillway capacity often much less even than the second column in Table 2. Only occasionally does such a dam fail; when it does, old inhabitants will say that the storm was the greatest in their memory of perhaps seventy years. Pondage saves many of them, but the really big floods, such as provide the controlling points on enveloping flood curves like those noted in Table 2, take a small but relentless toll of such dams when chance brings the intense rainfall to their particular valleys.

The subject of spillways is one to which the Society, as a body, could give more attention with advantage. It is not enough to list huge floods which would tear up valleys even if not augmented by dam failures. To provide spillways for such floods everywhere is not economically possible. Some principles as to the reasonable degree of chance to be taken might be enunciated. In the meantime, the adage of a hydraulic engineer of high repute to "make spillways as big as the Lord will let you" seems to be a good rule.

Water Purification

The war has not materially affected water purification, except to prevent the construction of needed improvements in a few cities. Water works chemicals generally have been sufficient for purification of the water. Aluminum sulfate is being manufactured from lower grades of bauxite, and while its coagulating property may not equal that of the aluminum sulfate formerly used, it is in approximate proportion to the iron and aluminum sulfate present. Water works are able to obtain sufficient chlorine for sterilization of the water.

Before war was declared there was a tendency toward higher chlorination of water. This trend has not been stopped entirely, and those places practicing higher chlorination in most instances are continuing with the treatment. The main advantage of the high chlorination is increased bacteriological removal, though improved taste is reported in some instances.

The addition of ammonia for partial dechlorination of the water following the addition of a high dosage of chlorine is being given some consideration. The highly polluted water on the South Side of Chicago, taken in from the lake at the Dunne and 68th Street cribs, was treated in this manner the last three months of 1942. In places where the treatment is being tried, there is no evidence at present that it produces better sterilization of the water than would be obtained by the addition of chlorine without ammonia in such amount that the water in the distribution system contained the same residual chlorine. Also, there is no evidence of much change in the taste of the water over straight chlorination.

The use of hexametaphosphate for corrosion prevention is increasing, though investigators are not in agreement as to whether it actually prevents corrosion or merely removes the corpus delicti. It may be noted that there is still a wide field open to water chemists in treating water so that corrosion, as manifested by reduced carrying capacity, will be slowed down at least. In the application of pipe cleaning methods there is great need of economical water treatment that will prevent rapid re-tuberculation after cleaning. With such treatment, pipe cleaning would become enormously more effective.

Effect of Copper, Brass, and Zinc on Humans.—The Committee would call attention to a series of scholarly articles by Dr. Frank E. Hale on this subject, published in *Water Works Engineering* dated January 28, February 11, February 25, and May 11, 1942.

Census of Water Treatment Plants.—A notable census of water treatment plants in the United States requiring more than two years was made by the U. S. Public Service beginning in 1939 and reported in the March 20, 1942, issue of Public Health Reports. A full summary appears in the December, 1942, *Journal* of the American Water Works Association. The detailed data of each treatment plant are published in the January 27, 1943, issue of *Water Works Engineering*. It appears that more than 74,000,000 out of a national population of about 131,700,000 enjoy water that has been treated. Statistics are available for 5,372 treatment plants serving about 62,500,000 people. About half of these plants, serving, however, only about 19,400,000 people, use chlorination as their principal treatment, generally with some other adjunctive treatment such as aeration, sedimentation, or coagulation and sedimentation. One hundred and thirty-eight plants serving about 500,000 employ some treatment other than filtration and do not use chlorine. About 49% of the 5,372 treatment plants of record receive their raw water from surface sources, 46% from ground-water sources, and the remaining 5% from a combination of ground and surface supplies; but the population served by surface sources is a higher percentage of the total—about 73%.

Committee on Survival and Retirement of Water Works Facilities

A Committee of the American Water Works Association on Survival and Retirement of Water Works Facilities, under the chairmanship of L. R. Howson, M. Am. Soc. C. E., has made a noteworthy progress report of statistical data on the life history of cast-iron pipe lines, valves, hydrants, and meters. One of its purposes is to furnish reliable data for the use of the Depreciation Committee of the National Association of Railroad and Utility Commissioners which is making a monumental report on Depreciation. The war is interfering with the progress of all this committee work but it has not been halted entirely.

F. A. Barbour L. R. Howson
J. R. Baylis J. A. Wade
W. W. Brush Thomas H. Wiggin, *Chairman*

Committee of the Sanitary Engineering Division on
Water Supply Engineering

January 21, 1943

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

CONFORMITY BETWEEN MODEL AND PROTOTYPE A SYMPOSIUM

Discussion

BY GLEN N. COX, M. AM. SOC. C. E.

GLEN N. COX,[19] M. AM. SOC. C. E.[19a]—The various papers of this Symposium are a valuable addition to the literature on the operation and reliability of hydraulic models.

The paper by Mr. Soucek is of special interest to the writer since it deals with a dam operating as a submerged weir. The definition of submergence quoted by Mr. Soucek infers that a weir can only be submerged if the elevation of the tailwater is above the crest of the weir. This assumption is satisfactory for any weir not having a sharp crest. However, a sharp-crested weir behaves as a submerged weir when the tailwater elevation is materially lower than the crest elevation. This is true because it is impossible to ventilate the underside of the nappe properly from the sharp-crested weir when the tailwater elevation approaches the crest elevation. A desirable definition of a submerged weir is as follows:

> A weir is said to be submerged whenever the tailwater rises to such an elevation that the discharge over the weir is affected.

Mr. Soucek is correct when he states that the discharge over an ogee crest is not materially affected when the submergence is less than 45%.

Mr. Soucek chose appropriate words in the statement (see heading, "Comment: Measurement of Tailwater Elevation")—

> "* * * the water level near the dam is at a lower elevation than farther downstream. The reason for a negative slope in this reach is a readjustment of velocity distribution which has an effect that is similar, as regards energy transfer, to the recovery of pressure head which occurs in the diffusing cone of a venturi meter."

NOTE.—This Symposium was published in October, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1942, by A. E. Niederhoff, Assoc. M. Am. Soc. C. E.; January, 1943, by Messrs. C. I. Grimm, and Joe W. Johnson; and February, 1943, by V. L. Streeter, Assoc. M. Am. Soc. C. E.

[19] Prof., Mechanics and Hydraulics, Louisiana State Univ., Baton Rouge, La.

[19a] Received by the Secretary February 15, 1943.

The downstream pressure should be measured at the point of minimum pressure, namely, at the throat of the venturi tube. The head recovered downstream from the throat of the venturi meter varies between rather wide limits and depends upon the original turbulence, the angle of divergence, the viscosity of the fluid, the roughness of the surface, and possibly upon other factors. Although it would be possible to calibrate a given tube and thus obtain a relationship between the discharge and over-all pressure drop for a given fluid, it would not be reasonable to expect that the same relationship would hold for the same tube with a different piping arrangement or in transporting a fluid of decidedly different physical properties. Neither could the calibration be used for tubes of other proportions and angles of divergence. On the other hand, calibrations that are based upon a downstream pressure reading taken at the throat can be used for predicting the discharge from other tubes and for other fluids flowing in the calibrated tube.

The same reasoning applies to the submerged weir. The magnitude of the recovery head downstream from the low pressure point depends upon the proportions and roughness of the channel downstream from the dam. No two natural channels could be expected to be similar in this reach, nor would the reaches upstream and downstream from the crest be similar. Fig. 67 shows the difference in percentage submergence when the tailwater elevation is measured near the dam as compared to the value when the tailwater elevation is measured at the point of maximum recovery. These values were measured upon a suppressed weir, there being no lateral expansion downstream from the crest. The transfer of energy is more efficient when the proportionate amount of the expansion is reduced. For these reasons, a consideration of model laws would reveal that it would be poor practice to include any more variables than necessary in the test arrangement and that, therefore, the tailwater elevation should be measured near the dam.

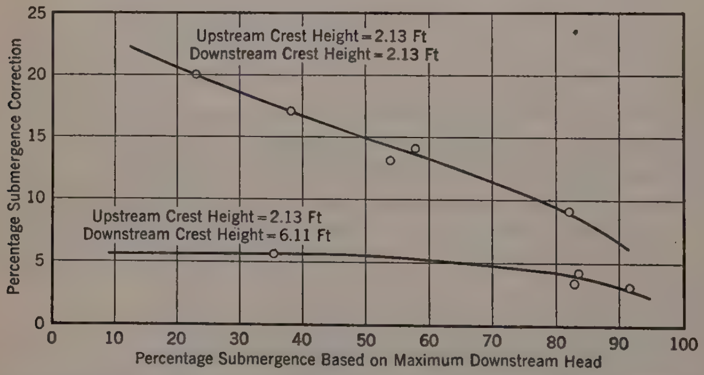

Fig. 67.—Correction Curve for Obtaining Percentage Submergence at Low Point Near Crest

Mr. Soucek referred to the maximum submergence for which the plunging nappe condition might be maintained. Fig. 68 shows a different method of presenting this subject. The values used in Fig. 68 were obtained from the Wisconsin study (45).[19b]

The discharge over an ogee, submerged weir can be represented by the formula

$$Q = C\,L\left(H + \frac{V^2}{2\,g}\right)^n \qquad (21a)$$

in which Q is the time rate of discharge, in cubic feet per second; L is the length of the crest, in feet; H is the measured head on the dam referred to the crest,

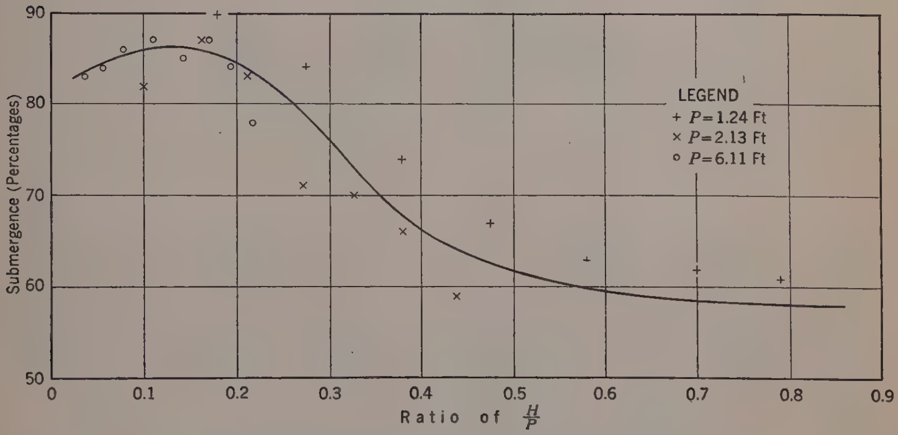

Fig. 68.—Maximum Submergence for Plunging Nappe Flow

in feet; V is the average upstream velocity of flow; g is the acceleration due to gravity; C is a coefficient; and n is an exponent. Both C and n depend upon the section of the crest, the type of flow, the submergence, and possibly other factors. The section of the ogee crest used for the Wisconsin tests is shown in

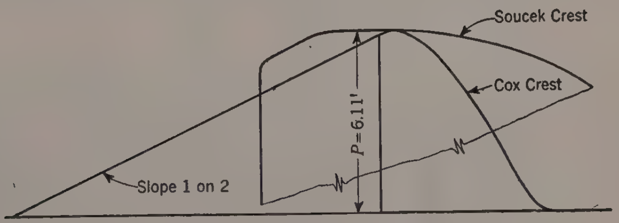

Fig. 69.—Comparative Crest Sections

Fig. 69, in which the section of the Soucek crest has been added for comparative purposes. The discharge equation for the Cox crest with the vertical back was

$$Q = 3.895\,L\left(H + \frac{V^2}{2\,g}\right)^{1.625} \qquad (21b)$$

[19b] Numerals in parentheses, thus: (45), refer to corresponding items in the Bibliography, which appears as the last unit of the Symposium, and at the end of discussion in this issue.

When the sloping approach was added to this same crest, the discharge equation became

$$Q = 3.725\, L \left(H + \frac{V^2}{2\, g} \right)^{1.596} \dots\dots\dots\dots (21c)$$

An analysis of the Soucek data provides the discharge equation,

$$Q = 3.1\, L \left(H + \frac{V^2}{2\, g} \right)^{1.55} \dots\dots\dots\dots (21d)$$

From Eqs. 21, it can be seen that both the coefficient and the exponent decrease as the crest is made relatively less sharp.

The submerged discharge equation (plunging nappe) for the Cox crest having the sloping approach was given by Eq. 21*a*, in which

$$C = 3.725 - \frac{0.193\, S^2}{1.15 - S} \dots\dots\dots\dots (22a)$$

$$n = 1.596 - \frac{0.035\, S^2}{2 - S} \dots\dots\dots\dots (22b)$$

and S is the submergence ratio based upon a tailwater reading taken near the crest. When $S = 0$, the values of C and n given by Eqs. 22 become the same as those given in Eq. 21*c*. A trial solution also shows only slight variation in C and n for values of S less than 0.5, which supports Mr. Soucek's conclusion that the submergence need not be considered for values of S less than 0.45.

TABLE 10.—SUBMERGED DAM BEHAVIOR

Adjusted Q (cu ft per sec)	Energy H (ft)	S (PERCENTAGES)		Indicated Q; model (cu ft per sec)	Q, Eq. 21d (cu ft per sec)	Difference, Cols. 1 and 6 (%)	Q, Eqs. 21a and 23 (cu ft per sec)	Difference, Cols. 5 and 8 (%)
		Observed	Corrected					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1,920	1.71	0	....	1,760	1,920	0	....	....
1,960	1.75	0	....	1,820	1,990	+ 1.5	....	....
2,190	1.78	0	....	1,860	2,040	− 6.9	....	....
2,260	1.88	0	....	2,050	2,220	− 1.8	....	....
2,410	1.97	0	....	2,210	2,390	− 0.8	....	....
2,700	2.14	0	....	2,530	2,720	+ 0.7	....	....
2,910	2.20	0	....	2,650	2,830	− 2.8	....	....
3,460	2.54	0	....	3,360	3,540	+ 2.3	....	....
3,830	2.67	0	....	3,640	3,820	− 0.3	....	....
4,490	3.00	0	....	4,380	4,580	+ 2.4	....	....
6,160	3.63	0	....	6,020	6,160	0	....	....
6,560	3.76	0	....	6,340	6,500	− 0.9	....	....
3,440	2.41	11.7	6.2	3,070	3,260	− 5.2	3,270	+6.5
9,650	4.71	40.8	35.3	9,270	9,200	− 4.7	9,090	−1.9
12,500	5.62	43.6	38.1	12,400	12,100	− 3.2	11,900	−4.0
12,500	5.38	47.2	41.8	11,500	11,300	− 9.6	11,100	−3.5
9,680	4.81	54.8	49.6	9,500	9,520	− 1.7	9,250	−2.6
3,470	2.67	57.4	52.2	3,620	3,840	+10.7	3,690	+1.9
15,800	6.29	60.0	54.9	14,700	14,400	− 8.9	13,800	−6.1
14,650	6.49	61.2	56.2	15,500	15,100	+ 3.1	14,400	−7.1
6,580	3.72	83.0	79.0	6,030	6,400	− 2.7	5,580	−7.5
6,890	4.05	87.5	84.0	6,700	7,300	+ 6.0	6,080	−9.3
7,340	4.13	89.2	86.0	6,740	7,520	+ 2.5	6,130	−9.1

The data contained in Table 9 have been computed to furnish Table 10. The discharges in Col. 6 have been obtained by use of Eq. 21*d*. Judging by the values in Col. 6, the Soucek crest experienced little, or no, diminution of discharge because of submergence. Also, the measured discharges for the higher submergences are in excess of the true discharges and to find a satisfying explanation for this conclusion becomes necessary. During the testing at Wisconsin (45), the writer observed that there was an upstream current near the floor of the channel on the upstream side of the weir for all flow conditions and that there was an upstream current near the floor of the channel on the downstream side for the flowing nappe type of flow. Mr. Soucek states that the gaging station was 470 ft downstream from the dam and that no gaging was taken when the ice jam was closer than 500 ft of the dam (see heading, "Observations on Prototype"). From this statement, it must be concluded that one or more gagings were made within 30 ft of the ice jam. The proximity of this barrier and the depth of the water on the upstream side of it certainly would indicate that an upstream current existed near the bed of the river at the gaging station. Such an upstream current could not have been detected by means of the Price current meter, which was used in making the gaging, and a computed discharge in excess of the true discharge would result from such an upstream current.

Should the same reducing factors for C and n be assumed to apply for the Soucek crest as did apply for the Cox crest with the approach, the following equations would result:

$$C = 3.1 - \frac{0.193\,S^2}{1.15 - S} \qquad \text{(23a)}$$

and

$$n = 1.55 - \frac{0.035\,S^2}{2 - S} \qquad \text{(23b)}$$

Eqs. 21*a* and 23 have been used to compute the discharge over the Soucek crest and the results appear in Table 10, Col. 8. These values have been compared with the discharges which were indicated by the model (Col. 5) to give Col. 9, Table 10. Considering the rather great difference in the section of the crests, the agreement between the values in Cols. 5 and 8 is remarkable, and it must be concluded that the model tests were conducted very carefully by Mr. Soucek.

Bibliography.—

(45) **"The Submerged Weir as a Measuring Device,"** by G. N. Cox, *Bulletin No. 67*, Univ. of Wisconsin Eng. Experiment Station, Madison, Wis., 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

AERATION OF SPILLWAYS

Discussion

By Joe W. Johnson, Assoc. M. Am. Soc. C. E.

Joe W. Johnson,[4] Assoc. M. Am. Soc. C. E.[4a]—Air is withdrawn from the underside of a nappe by the falling sheet acting as a friction pump. The air withdrawn is replaced from two sources: (1) The air that flows through air vents to the space under the nappe, and (2) that which, upon being drawn into the downstream pool by both sides of the falling sheet, strikes the channel bottom and circulates back and upward to the space under the nappe. The observations[5] of the writer on a 1-ft weir indicated that the quantity of air withdrawn was proportional to the velocity of the falling sheet or, within certain limits, to the discharge. The only effect of the position of the downstream water level is the variation of the quantity of air circulated. If a deflector is placed near the underside of the nappe, as shown by Mr. Hickox in Fig. 3(*b*) and by the writer in Fig. 11, air is prevented from circulating backward into the space under the nappe. Therefore, all air that is finally withdrawn must first enter through the air vents. A direct measure of the power of the falling sheet to withdraw air is thus determined by the insertion of a deflector. The data from the writer's experiments on a 1-ft weir with heads never exceeding 0.4 ft are shown in Table 1(*a*), tests 76 and 83 to 106. A plot of these data is shown in Fig. 11 from which Eq. 1 is obtained.

In passing, it may be well to discuss briefly the reason for the differences between Eqs. 1 and 2. Upon examination of these equations, the reader is inclined to conclude that a "break" occurs in the relationship between q_a and q_o at a head of 0.4 ft. Actually, Eq. 1 was based on only the laboratory experiments plotted in Fig. 11, and Eq. 2 was based on both the laboratory experiments and some data on a large weir in the San Joaquin Valley. The extensive data collected since the writer's experiments have permitted a conclusive analysis of the problem of partial aeration as well as the design of air vents. Thus, by utilizing all available data from both the laboratory and the field,

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Note.—This paper by G. H. Hickox, M. Am. Soc. C. E., was published in December, 1942, *Proceedings*.

[4] Asst. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

[4a] Received by the Secretary January 26, 1943.

[5] "The Aeration of Sharp-Crested Weirs," by Joe W. Johnson, a thesis presented to the Univ. of California, Berkeley, Calif., in May, 1934, in partial fulfilment of the requirements for the degree of Master of Science in Engineering (unpublished).

Mr. Hickox has obtained excellent correlation between the various observations by means of dimensionless groups. In view of the wide range of conditions covered by this analysis, the author's procedure for designing air vents should be considered as superseding all previous methods.

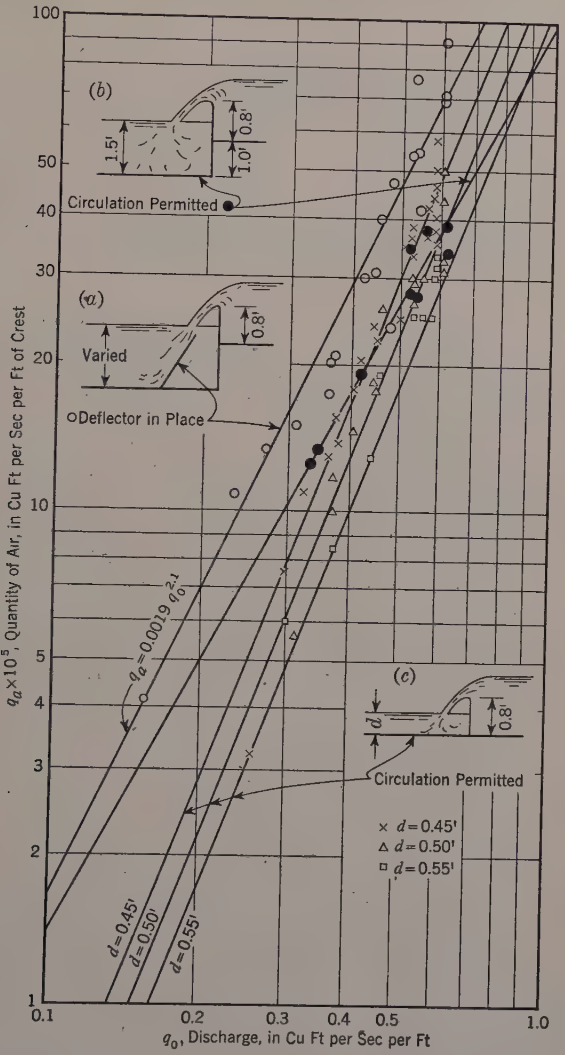

FIG. 11.—RELATION BETWEEN DISCHARGE AND QUANTITY OF AIR DRAWN THROUGH INLETS TO THE UNDERSIDE OF THE NAPPE OF A SHARP-CRESTED WEIR WITH VARIOUS CONDITIONS OF BOTTOM POSITION AND DEPTH IN THE DOWNSTREAM POOL

To show the effect of different positions of the downstream channel bed and the effect of different depths of pools, several experiments were performed where conditions existed as shown in Figs. 11(*b*) and 11(*c*). The quantity of air, as indicated by the various curves, is that delivered beneath the nappe through the air vents. Fig. 11 shows that, for a particular position of the downstream channel bed, the volume of air taken in through the vents increases as the depth of pool decreases. In other words, the volume of air circulated decreases as the depth of pool is decreased since the volume drawn outward is the same for all depths.

When a deflector is placed to prevent air from entering into the space under the nappe by any method except through the vents, the quantity of air withdrawn is given by Eq. 9, which is based on all available laboratory and field data. However, if air is allowed also to enter the underside of the nappe by circulation, less air is taken in through the vents and the relationship between flow of air through the vents and the other variables is different from that shown by Eq. 9. The quantity of air flowing through the vents depends on the position of the bed and the depth of the downstream pool with respect to the weir crest.

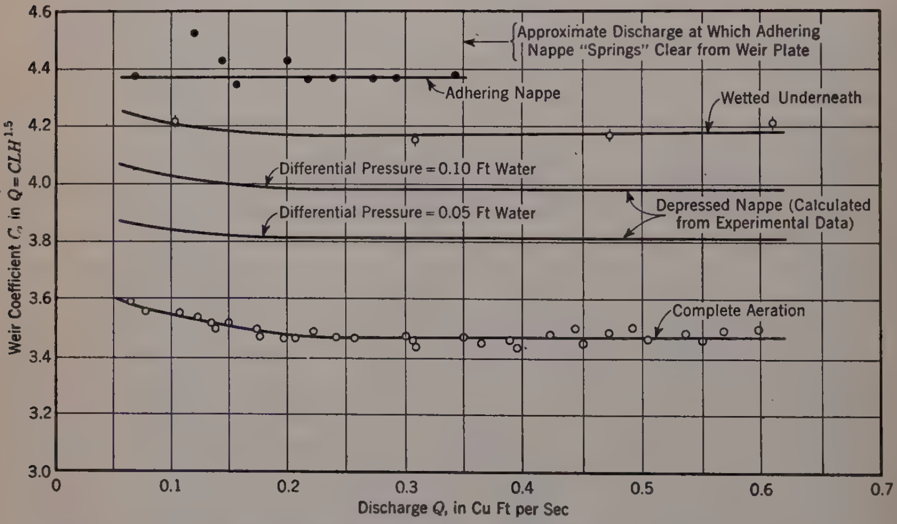

FIG. 12.—WEIR COEFFICIENT FOR VARIOUS CONDITIONS OF AERATION

A reduction in the pressure under the nappe depresses the nappe inward and reduces the head on the weir by a certain amount. The effect of depressing the nappe generally expands the cross-section area of the nappe at the crest and thus increases the discharge by making the head more effective. The net result is to increase the weir coefficient.[6] For example, Fig. 12 shows the variation of the weir coefficient with the degree of aeration for the 1-ft weir tested by the writer. For a certain head the effect of preventing free access of air to the underside of the nappe gives a discharge greater than that obtained by complete aeration. A maximum increase in discharge of 23% resulted with an "adhering" nappe, and the "wetted underneath" and "depressed" conditions gave increases of 18% and 6%, respectively.

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[6] "Hydraulic Laboratory Practice," by John R. Freeman, A.S.M.E., 1929, p. 147.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

THE GREASE PROBLEM IN SEWAGE TREATMENT

Discussion

BY ALMON L. FALES AND SAMUEL A. GREELEY,
MEMBERS, AM. SOC. C. E.

ALMON L. FALES[17] AND SAMUEL A. GREELEY,[18] MEMBERS, AM. SOC. C. E.[18a] —Mr. Weston's discussion of this paper is the result of a large amount of analytical work and experimentation based on a forward looking conception of the problem in Boston Harbor. The data presented illustrate the elusiveness of the grease problem. Table 8, for instance, indicates a near-maximum percentage removal of fats with a 4-hr detention period. For large installations this is a relatively long period and perhaps was one of the reasons for the experiments in stirring, aeration, return sludge, chlorine, and salt. The discussion indicates a leaning toward plain sedimentation for the removal of fats.

Mr. Ludwig has presented an up-to-date discussion regarding the analytical determination of grease. His statement that the determination of grease in the laboratory should be related to design and operation, and should indicate substances of sanitary significance, is a good suggestion for future consideration. It might be well to realize that often much grease is removed with the sludge as well as with the scum. In general, this discussion prompts the suggestion by the writers that operating records could well be kept in the form of a grease balance in which the grease in the raw sewage would be distributed to the grease recovered in scum and sludge plus the grease in the effluent.

Note is made of the discussion by Mr. Mahlie which prompts the suggestion by the writers that those studying the grease problem should refer to his excellent paper (13).[18b]

The discussion by Mr. Dickinson states that the practice of burying grease cannot be condemned too strongly. In the opinion of the writers this state-

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NOTE.—This paper by Almon L. Fales and Samuel A. Greeley, Members, Am. Soc. C. E., was published in February, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by Arthur D. Weston, M. Am. Soc. C. E.; May, 1942, by Harvey F. Ludwig, Jun. Am. Soc. C. E.; June, 1942, by W. S. Mahlie, Esq.; September, 1942, by Messrs. Denis Dickinson, and Charles R. Velzy; and November, 1942, *Proceedings*, by F. M. Dawson, M. Am. Soc. C. E.

[17] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[18] (Greeley & Hansen), Chicago, Ill.

[18a] Received by the Secretary February 1, 1943.

[18b] Numerals in parentheses, thus: (13), refer to corresponding items in the Bibliography of the paper (see Appendix).

ment does not apply to the smaller sewage treatment plants with sufficient adjacent land area for disposal by burial. This is a simple method which avoids some of the operating problems of grease disposal in larger plants.

Such operating problems are well described in the discussion by Mr. Velzy. The removal of greasy scum from sedimentation tanks, the operation of pumps for handling the skimmings, and the operations required to keep scum and sludge discharge pipes free of grease, are typical problems requiring attention in design. The formation of scum in sludge digestion tanks is another operating problem. Such problems are accentuated by the presence of small pieces of trash in the scum and sludge which Mr. Velzy indicates can be reduced by good screening. Another problem discussed by Mr. Velzy is the control of temperature in burning greasy scum. These problems indicate that operating analyses or research studies might include determinations of viscosity, congealing characteristics, and the Btu-content.

Dean Dawson calls attention to the grease interceptor which is a device for removing grease at the source of the sewage. The writers doubt that such a method has general applicability in municipal practice, although the device is applicable at places where relatively large amounts of grease are likely to enter the sewers; as, for instance, garages and large kitchens, such as those at hotels and restaurants. Such interceptors appear also to have a place at the relatively large kitchens of army cantonments. Several municipalities have adopted ordinances requiring installation of interceptors at garages and large kitchens. In general, when the disposal of the intercepted grease is difficult, the maintenance of interceptors requires careful and thorough inspection.

Since February, 1942, when the paper was published in *Proceedings*, considerable progress has been made toward a standard method for the determination of grease in sewage and it may be that the next edition of "Standard Methods for the Examination of Water and Sewage" (1) will present such a method. The war year, however, has tended to retard the accumulation of data, on an operating scale, regarding grease in sewage, effluents, and sludge.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

NUMERICAL PROCEDURE FOR COMPUTING DEFLECTIONS, MOMENTS, AND BUCKLING LOADS

Discussion

BY ROBERT A. WILLIAMSON, JUN. AM. SOC. C. E.

ROBERT A. WILLIAMSON,[33] JUN. AM. SOC. C. E.[33a]—In members subjected to combined axial and bending loads (commonly called beam columns) the effect of secondary moments caused by end thrusts cannot be ignored safely when the compressive load is any appreciable percentage of the critical buckling load. This is plainly demonstrated by the results of Fig. 17.

When the beam column is one of a series of members comprising a continuous structure, the values of the elastic constants and fixed-end moments required for the usual moment-distribution analysis depend, in part, on the magnitude and sign of the axial load.

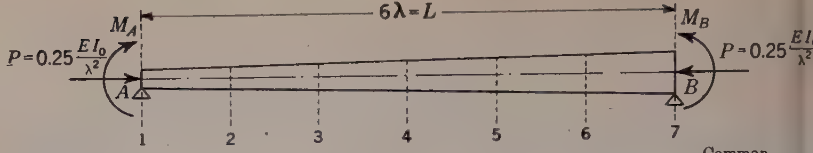
For the case of variable section, evaluation of these quantities is greatly facilitated by the use of Professor Newmark's procedure, details of which are shown in Figs. 41 to 44, inclusive, using as an example the beam of Figs. 10 and 11 subjected to an axial compressive load.

Determination of the elastic constants requires the computation of the end slopes at A and B (Fig. 41) due to end moments applied separately, first at A, then at B, including the effect of the given axial load, P, in both cases, assuming the beam to be simply supported. Initial deflections, w_i, and corresponding end slopes were obtained from Figs. 10(*c*) and 11(*b*). The first trial is shown in detail for end moment M_A, the intermediate trials being omitted. Final results only are shown for end moment M_B.

NOTE.—This paper by N. M. Newmark, Assoc. M. Am. Soc. C. E., was published in May, 1942 *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1942, by Bruce Johnston, Assoc. M. Am. Soc. C. E.; June, 1942, by M. S. Ketchum, Jr., Assoc. M. Am. Soc. C. E.; September, 1942, by Messrs. John B. Wilbur, Ralph W. Stewart, and Stefan J. Fraenkel; October, 1942, by Alfred S. Niles, Assoc. M. Am. Soc. C. E.; November, 1942, by Camillo Weiss, M. Am. Soc. C. E.; and January, 1943, by Messrs. A. A. Eremin, and Myron L. Gossard.

[33] Stress Analyst, Vega Aircraft Corp., Burbank, Calif.

[33a] Received by the Secretary February 1, 1943.



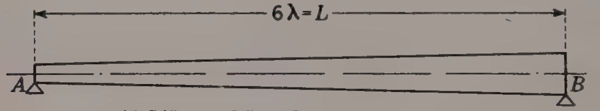
(a) Deflections and End Slopes for $M_A = 180$, $P = 0.25\frac{E\,I_o}{\lambda^2}$

	1		2		3		4		5		6		7	Common Factors
I/I_o														
w_i for $M_A = 180$ From Fig. 10(c)	0		110.6		141.4		130.8		97.1		51.1		0	$\lambda^2/(E\,I_o)$
Assumed w_a	0		34.0		47.0		44.0		36.0		19.0		0	$\lambda^2/(E\,I_o)$
$w_o = w_i + w_a$	0		144.6		188.4		174.8		133.1		70.1		0	$\lambda^2/(E\,I_o)$
Moments due to P	0		36.2		47.2		43.6		33.3		17.5		0	1
Distributed Angle Changes	0		−18.1		−15.7		−10.9		−6.7		−2.9			$1/(E\,I_o)$
Assumed Average Slope, Part (1)		38.1		20.0		4.3		−6.6		−13.3		−16.2		$\lambda/(E\,I_o)$
Trial Deflection, Part (1)	0		38.1		58.1		62.4		55.8		42.5		26.3	$\lambda^2/(E\,I_o)$
Deflection, Part (2)	0		−1.5		−1.3		−0.9		−0.6		−0.2		0	$\lambda^2/(E\,I_o)$
Linear Correction to Deflection	0		−4.4		−8.8		−13.2		−17.5		−21.9		−26.3	$\lambda^2/(E\,I_o)$
Resultant Deflection, w'_a	0		32.2		48.0		48.3		37.7		20.4		0	$\lambda^2/(E\,I_o)$
Final Deflections w_a Giving Same Deflections w'_a	0		32.4		48.4		48.9		38.1		20.6		0	$\lambda^2/(E\,I_o)$
Final w_o	0		143.0		189.8		179.7		135.2		71.7		0	$\lambda^2/(E\,I_o)$
Final Moments	180		185.7		167.4		134.9		93.8		47.9		0	1
Final End Slopes	212.9												−73.0	$\lambda/(E\,I_o)$

(b) Deflections and End Slopes for $M_B = 180$, $P = 0.25\frac{E\,I_o}{\lambda^2}$

	1	2	3	4	5	6	7	Common Factors
Final Deflections w_a Giving Same Deflections w'_a	0	19.1	30.4	32.4	26.4	14.7	0	$\lambda^2/(E\,I_o)$
Final w_o	0	67.6	113.1	129.4	115.4	71.7	0	$\lambda^2/(E\,I_o)$
Final Moments	0	46.9	88.2	122.3	148.8	167.9	180	1
Final End Slopes	72.7						−84.9	$\lambda/(E\,I_o)$

FIG. 41.—DEFLECTIONS AND BENDING MOMENTS FOR END MOMENTS COMBINED WITH AXIAL COMPRESSION



(a) Stiffness and Carry-Over Factors for Left End

			Common Factors
212.9	End Slopes, $M_A = 180$	−73.0	$\lambda/(E\,I_o)$
−62.5	End Slopes, $M_B = -0.861 \times 180$	73.0	$\lambda/(E\,I_o)$
150.4	Total Slope, $M_A = 180$, $M_B = -155$	0	$\lambda/(E\,I_o)$

Carry-Over Factor $= -0.861 = C_A$

$$\text{Stiffness} = \frac{180}{150.4}\frac{E\,I_o}{\lambda} = 1.195\frac{E\,I_o}{\lambda} = K_A$$

(b) Stiffness and Carry-Over Factors for Right End

72.7	End Slopes, $M_B = 180$	−84.9	$\lambda/(E\,I_o)$
−72.7	End Slopes, $M_A = -0.341 \times 180$	24.9	$\lambda/(E\,I_o)$
0	Total Slope, $M_B = 180$, $M_A = -61.5$	−60.0	$\lambda/(E\,I_o)$

Carry-Over Factor $= -0.341 = C_B$

$$\text{Stiffness} = \frac{180}{60.0}\frac{E\,I_o}{\lambda} = 3.00\frac{E\,I_o}{\lambda} = K_B$$

(c) Rigidity Factor
End Moments from Settlement, Δ, of End B,
A and B Restrained from Rotation

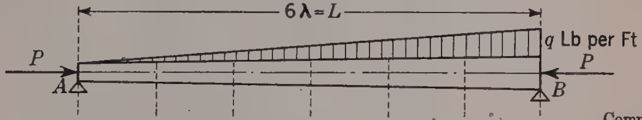
$$-0.371\frac{E\,I_o\Delta}{\lambda^2} = K_A(1 + C_A)\frac{\Delta}{6\lambda^2} \qquad K_B(1 + C_B)\frac{\Delta}{6\lambda^2} = 0.671\frac{E\,I_o\Delta}{\lambda^2}$$

$$\text{Rigidity} = \text{Shear when } \Delta = 1 = \frac{(0.371 + 0.671)}{6\lambda}\frac{E\,I_o}{\lambda^2} = 0.1739\frac{E\,I_o}{\lambda^3}$$

FIG. 42.—DETERMINATION OF ELASTIC CONSTANTS

Fig. 42 illustrates the determination of stiffness and carry-over factors and rigidity from the results of Fig. 41 by a method similar to that of Fig. 11(*c*). (The formula for end moments in Fig. 42(*c*) is easily derived.[34])

For determining fixed-end moments the additional quantities needed are the end slopes of the beam, assumed simply supported and subjected to the given lateral and axial load. Fig. 43 shows the calculation of these values for a triangular lateral loading condition, intermediate trials being omitted.



I/I_o	1		2		3		4		5		6		7	Common Factors
(*a*) Deflections for Triangular Load with $P = 0$														
Ordinates to Load Diagram	0		−30		−60		−90		−120		−150		−180	$q/180$
Assumed Average Shear, Part (1)		180		150		90		0		−120		−270		$q\lambda/180$
Trial Moments, Parts (1) and (2)	0		180		330		420		420		300		30	$q\lambda^2/180$
Linear Correction to Moments	0		−5		−10		−15		−20		−25		−30	$q\lambda^2/180$
Ordinates of Moment Diagram	0		175		320		405		400		275		0	$q\lambda^2/180$
Ordinates to Angle-Change Diagram	0		−87.5		−106.7		−101.3		−80.0		−45.8		0	$q\lambda^2/(180\,E\,I_o)$
Assumed Average Slope, Part (1)		237.5		150.0		43.3		−58.0		−138.0		−183.8		$q\lambda^3/(180\,E\,I_o)$
Trial Deflections, Part (1)	0		237.5		387.5		430.8		372.8		234.8		51.0	$q\lambda^4/(180\,E\,I_o)$
Deflections, Part (2)	0		−7.3		−8.9		−8.4		−6.7		−3.8		0	$q\lambda^4/(180\,E\,I_o)$
Linear Correction to Deflections	0		−8.5		−17.0		−25.5		−34.0		−42.5		−51.0	$q\lambda^4/(180\,E\,I_o)$
Deflections, w_i	0		221.7		361.6		396.9		332.1		188.5		0	$q\lambda^4/(180\,E\,I_o)$
End Slopes	239.1=221.7+17.4												−188.5−8.1=−196.6	$q\lambda^3/(180\,E\,I_o)$
(*b*) Deflections for Triangular Load with $P = 0.25\frac{E\,I_o}{\lambda^2}$														
Deflections, w_i	0		110.9		180.8		198.5		166.1		94.3		0	$q\lambda^4/(90\,E\,I_o)$
Assumed w_a	0		38.1		62.1		68.4		57.2		32.4		0	$q\lambda^4/(90\,E\,I_o)$
$w_o = w_i + w_a$	0		149.0		242.9		266.9		223.3		126.7		0	$q\lambda^4/(90\,E\,I_o)$
Moments due to P	0		37.3		60.8		66.9		55.9		31.7			$q\lambda^2/90$
Distributed Angle Changes	0		−18.7		−20.3		−16.7		−11.2		−5.3		0	$q\lambda^2/(90\,E\,I_o)$
Assumed Average Slope, Part (1)		38.7		20.0		−0.3		−17.0		−28.2		−33.5		$q\lambda^3/(90\,E\,I_o)$
Trial Deflections, Part (1)	0		38.7		58.7		58.4		41.4		13.2		−20.3	$q\lambda^4/(90\,E\,I_o)$
Deflections, Part (2)	0		−1.6		−1.7		−1.4		−0.9		−0.4		0	$q\lambda^4/(90\,E\,I_o)$
Linear Correction to Deflections	0		3.4		6.8		10.2		13.5		16.9		20.3	$q\lambda^4/(90\,E\,I_o)$
Deflections, w_a	0		40.5		63.8		67.2		54.0		29.7		0	$q\lambda^4/(90\,E\,I_o)$
Final Deflections w_a Giving Same Deflections w'_a	0		40.7		63.8		67.0		53.8		29.5		0	$q\lambda^4/(90\,E\,I_o)$
Final w_o	0		151.6		244.6		265.5		219.9		123.8		0	$q\lambda^4/(90\,E\,I_o)$
Final Moments	0		125.4		221.3		269.0		255.0		168.4		0	$q\lambda^4/90$
Final End Slopes	164.2												−128.6	$q\lambda^3/(90\,E\,I_o)$

FIG. 43.—DEFLECTIONS AND BENDING MOMENTS FOR TRIANGULAR LOAD COMBINED WITH AXIAL COMPRESSION

The computation of fixed-end moments, shown in Fig. 44(*a*), utilizes the data of Figs. 42(*a*), 42(*b*), and 43 and the same general principles previously applied in the determination of the elastic constants. In Fig. 44(*b*) the resultant deflections and bending moments for fixed ends and axial load are tabulated.

As a check, the bending moments of Fig. 44(*b*) are used to obtain the results of Fig. 44(*c*), the computed deflections and end slopes differing from those of Fig. 44(*b*) by a maximum of about 2%. Much of the work was done with a 5-in. slide rule, the remainder with a 10-in. slide rule.

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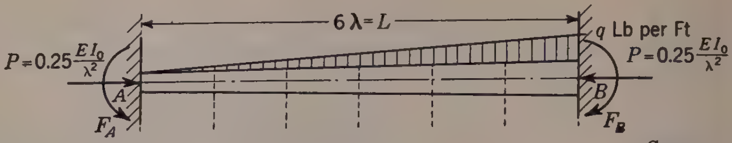
[34] "One Story Frames Analyzed by Moment Distribution," *Concrete Information Bulletin No. S T 42*, Portland Cement Assn., April, 1941, pp. 8 and 9.

In all cases where the value of w_a for the first trial was computed from Eq. 4, a total of two trials gave a value of w'_a so close to the value of w'_a from a third trial that the extra trial was unnecessary. The effect of omitting deflections w_a, computed from Eq. 4, was to increase the required number of trials to three.


$6\lambda = L$
q Lb per Ft
$P = 0.25 \frac{EI_0}{\lambda^2}$
$P = 0.25 \frac{EI_0}{\lambda^2}$
A
B
F_A
F_B

Common Factors

(a) Fixed-End Moments

End Slopes, Beam Simply Supported (Fig. 43(b))

1.824 $\quad - 1.427$ $\quad q\lambda^3/(E I_o)$

End Rotation Producing Fixity at A, with B Clamped

$- 1.824$ $\quad 0$ $\quad q\lambda^3/(E I_o)$

Resulting End Moments

$- 2.180\, q\lambda^2 = 1.195 \frac{E I_o}{\lambda} \left(- 1.824 \frac{q\lambda^3}{E I_o} \right)$ $\quad - 2.180\, q\lambda^2 (- 0.861) = 1.879\, q\lambda^2$

End Rotation Producing Fixity at B, with A Clamped

0 $\quad 1.427$ $\quad q\lambda^3/(E I_o)$

Resulting End Moments

$1.460\, q\lambda^2 = - 4.281\, q\lambda^2 (- 0.341)$ $\quad 3.00 \frac{E I_o}{\lambda} \left(1.427 \frac{\lambda^3}{E I_o} \right) = - 4.281\, q\lambda^3$

Fixed-End Moments

$- 0.720\, q\lambda^2 = F_A$ $\quad F_B = - 2.402\, q\lambda^2$

(b) Resultant Deflections and Bending Moments
$P = 0.25\, E I_o/\lambda^2$ in All Cases

Deflections due to F_A	0	−103	−137	−130	−98	−52	0	$q\lambda^4/(180\, E I_o)$
Deflections due to F_B	0	−163	−272	−311	−278	−173	0	$q\lambda^4/(180\, E I_o)$
Deflections due to $F_A + F_B$	0	−266	−409	−441	−376	−225	0	$q\lambda^4/(180\, E I_o)$
Deflections due to Triangular Load	0	303	489	531	440	248	0	$q\lambda^4/(180\, E I_o)$
Resultant Deflections	0	37	80	90	64	23	0	$q\lambda^4/(180\, E I_o)$
Resultant End Slopes	0						0	$q\lambda^3/(180\, E I_o)$
Moments due to F_A	−130	−134	−121	−98	−68	−35	0	$q\lambda^2/180$
Moments due to F_B	0	−113	−212	−294	−358	−404	−433	$q\lambda^2/180$
Moments due to $F_A + F_B$	0	−248	−333	−392	−426	−440	−433	$q\lambda^2/180$
Moments due to Triangular Load	0	250	443	538	510	337	0	$q\lambda^2/180$
Resultant Moments	−130	2	110	146	84	−103	−433	$q\lambda^2/180$

(c) Check Values of Deflections and End Slopes
Obtained from Above Moments

Deflections	0	36.2	79.4	88.8	63.5	22.6	0	$q\lambda^4/(180\, E I_o)$
End Slopes	−3.0						−0.9	$q\lambda^3/(180\, E I_o)$

FIG. 44.—FIXED-END MOMENTS, DEFLECTIONS, AND BENDING MOMENTS FOR TRIANGULAR LOAD COMBINED WITH AXIAL COMPRESSION

In comparison with other procedures devised for this type of problem, Professor Newmark's method affects savings in time and labor, gives equally good results, and is more easily checked.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

MECHANICS OF CREEP FOR STRUCTURAL ANALYSIS

Discussion

BY JOSEPH MARIN, ASSOC. M. AM. SOC. C. E.

JOSEPH MARIN,[30] ASSOC. M. AM. SOC. C. E.[30a]—The writer is greatly indebted to Professor Hrennikoff for pointing out two errors in this paper. With reference to the first one, it appears that the limitation in using only odd integer values for n is not a serious one. This fact is realized when the approximations involved in formulating a creep-stress law are considered and when the necessity of extrapolation is noted. If the test data on the material warrant a more accurate analysis than that indicated by the use of odd integers for n, it is always possible to solve the problem for two successive odd values of n and to interpolate for a particular value of n. Regarding the second error noted by Professor Hrennikoff, the writer realized that the law of superposition does not apply, and noted this in the paper (see heading, "Statically Indeterminate Structures: Example 5"). The general theory given in the paper ("Statically Indeterminate Structures: Moment-Distribution Method") appears to the writer to be correct. The application of this theory in Example 5 is incorrect as Professor Hrennikoff points out. That is, a new set of values for the carry-over and distribution factors should have been determined at each step in the moment-distribution procedure.

The comments of Mr. Turner are of considerable interest and importance in the general problem of behavior of materials subjected to stresses. The validity of the assumption that plane sections remain plane is based on creep-deformation measurements made on beam specimens.[3]

The discussion by Mr. Stewart regarding the justification for considering creep and the difficulties encountered in applying creep-stress analysis to

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NOTE.—This paper by Joseph Marin, Assoc. M. Am. Soc. C. E., was published in May, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1942, by Messrs. A. Hrennikoff, C. A. P. Turner, and Ralph W. Stewart.

[30] Prof., Eng. Mechanics, Pennsylvania State College, State College, Pa.

[30a] Received by the Secretary January 14, 1943.

[3] "Creep of Aluminum Subjected to Bending at Normal Temperatures," by J. Marin and L. E. Zwissler, *Proceedings*, A.S.T.M., Vol. 40, 1940, p. 937.

structures is significant. In many problems, creep may not be important, and in some cases it may be desirable to develop methods of analysis. The use of the words "basic" and "fundamental" seem to the writer questions of definition and point of view. However, the analysis given in Fig. 15 has considerable merit. This analysis, applied to the case of creep, gives similar results to Example 5 and is therefore incorrect.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

RELATION OF UNDISTURBED SAMPLING TO LABORATORY TESTING

Discussion

BY JACOB FELD, M. AM. SOC. C. E.

JACOB FELD,[5] M. AM. SOC. C. E.[5a]—The careful analysis of the problem of undisturbed soil samples, in which the author lists proof of each point, is a contribution to the subject of soil mechanics. The identification of clays by the shape of the stress-strain curve may well be considered a means of classification sufficiently accurate to be used as a design tool. Many structures are planned to be located on clay soils, where the cost of expensive sampling and testing has not been warranted. If the engineer is certain that the samples and the tests will indicate correctly the future action of the structure, he might be justified in ordering the expenditure. The use of full-scale data from previous experience (and troubles) with foundations as a basis of design of other structures is not feasible without a guarantee that the soils encountered are similar. The author's method can be used to prove similarity, even without the exact sampling procedure.

The action of clay soils under loadings, in test samples and in nature, can be described by an analogy. Assume the existence of a large continuous bed of sponge living at the bottom of the sea. The density of the sponge is the result of the hydrostatic pressure above it, within it, and around it, together with its skeleton strength. Change in water depth will affect the density immediately, subject to rebound with time as the internal water pressure (from osmosis) changes to balance the new loading. A load imposed on the sponge bed will settle, immediately due to compression of the skeleton and of the water and additionally as the water under the load is pressed outward, to the extent that the adjacent sponge will absorb the surplus. The volume change in the sponge under the load will affect the adjacent materials.

Excavating a test pit for determining sponge conditions at any depth is likely to yield unreliable results, because of the expansion at the new surface

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NOTE.—This paper by P. C. Rutledge, Assoc. M. Am. Soc. C. E., was published in November, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1942, by Messrs. Benjamin K. Hough, Jr., and F. M. Van Auken.

[5] Cons. Engr., New York, N. Y.

[5a] Received by the Secretary February 6, 1943.

as soon as the pit is dug. The job of removing an "undisturbed" sample for testing, without changing the physical characteristics, is almost hopeless. However, if a sample can be subjected to approximately the same exterior forces and moisture conditions in a laboratory test, its natural reactions under new loading can be determined. This paper indicates how one can be assured that the laboratory procedure reproduces original or natural conditions (natural void ratio and preconsolidation load). Such assurance will go far toward instilling confidence in the value of soil sampling and of soil testing.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

ENTRAINMENT OF AIR IN FLOWING WATER A SYMPOSIUM

Discussion

BY MESSRS. ROBERT T. KNAPP, AND CARL E. KINDSVATER

ROBERT T. KNAPP,[26] M. AM. SOC. C. E.[26a]—The entrainment of air by high-velocity flows in open channels is an important phenomenon about which too little is known. Mr. Hall is to be congratulated for his contribution, both from the observational and analytical standpoints. He deals not only with the air entrainment problem, but also with the general problem of high-velocity flow around horizontal and vertical curves in open channels. The writer wishes to comment briefly upon, first, the flow around horizontal curves, and, second, the problem of air entrainment through the free surface of the flow.

High-Velocity Flow around Horizontal Curves.—Until quite recently, little attention has been given to curvilinear flow in high-velocity open channels. However, in the past few years considerable work has been done in this field.[27] Since Mr. Hall has not presented the concepts of this regimen of flow that have been found useful, a brief review seems to be in order.

When applied to the flow of a liquid in an open channel, the term, "high-velocity flow," means flow at a velocity greater than the critical, or the wave velocity for that particular depth. The velocity of a surface wave is the velocity of propagation of a disturbance; that is, a force through the flow. In flow around a horizontal curve in an open channel ("horizontal" is used loosely in the sense of being "in the plane of the undisturbed surface"), the walls of the channel force the adjacent layers of liquid to move in curved paths. In flow that is slower than the wave velocity, these forces are transmitted across and along the channel at a speed sufficient to permit equilibrium to be established between the centrifugal forces, resulting from the curved path, and the

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NOTE.—This Symposium was published in September, 1942, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: January, 1943, by Warren DeLapp, Jun. Am. Soc. C. E.; and February, 1943, by Karl R. Kennison, M. Am. Soc. C. E.

[26] Associate Prof., Hydr. Eng., California Inst. of Technology, and Cooperative Agent, SCS., Pasadena, Calif.

[26a] Received by the Secretary, January 29, 1943.

[27] "Curvilinear Flow of Liquids with Free Surfaces at Velocities Above that of Wave Propagation," by Robert T. Knapp and Arthur T. Ippen, *Proceedings*, Fifth International Cong. for Applied Mechanics, 1938, p. 531.

lateral pressure gradient which manifests itself as the banking of the free surface. This equilibrium is seen to be attained at a cross slope of $\tan \psi = \frac{V^2}{g\,r}$ (Eq. 32). Unfortunately, this equilibrium condition does not hold for conditions of supercritical flow. The reason for this is clear. In low-velocity curvilinear flow two forces act on each liquid element to cause it to travel in curved paths. They are an "over-force" from the superelevation on the outer wall, and an "under-force" from the depression on the inside wall. This is possible only because the wall effect travels faster than the flow. In supercritical flow this state of affairs no longer exists. The effect of the change of direction of the wall is no longer felt directly across the channel but propagates laterally only at the wave velocity C. Thus it crosses the channel at an angle

$$\beta = \sin^{-1} \frac{C}{V} \dots\dots\dots\dots (46)$$

which means, if b is the width of the channel, that the effect reaches the opposite side first at a distance downstream of

$$L = \frac{b}{\tan \beta} \dots\dots\dots\dots (47)$$

For a channel whose mean radius of curvature is r, this corresponds to an angular distance

$$\theta_o = \tan^{-1} \frac{b}{\left(r + \frac{b}{2}\right) \tan \beta} \dots\dots\dots\dots (48)$$

In this distance all of the force required to cause the water to follow the curved path was supplied by one wall instead of by equal forces from both walls. Therefore, the pressure rise (the rise in the water surface at the outside wall) keeps increasing until it is about twice as high as it would be for a flow at the same velocity, but with a depth sufficient to cause the critical velocity to be above the velocity of flow. This maximum pressure rise corresponds to a cross slope, $\psi_{s,c}$, which is twice that given by Eq. 32:

$$\tan \psi_{s,c} = \frac{2\,V^2}{g\,r} \dots\dots\dots\dots (49)$$

However, below the point at which the effect from the opposite wall has succeeded in crossing the channel (Eqs. 47 and 48), the superelevation is obviously too great; so it decreases, leaving a maximum at this point. The subsequent history is easily visualized by considering that the behavior of the surface is thought of as a wave train oscillating about the equilibrium depth, given by Eq. 32 with the oscillation having a wave length of $2\,\theta_o$ and an amplitude of $\frac{V^2\,b}{2\,g\,r}$. The alternate maxima and minima are formed on both the outer and inner walls. The disturbance train persists into the downstream tangent with a magnitude depending upon the relative position of the last maximum in the curve. This behavior in the curve is clearly shown in Fig. 8, in which the last maximum is on the outer wall, apparently near the end of curvature. In Fig. 7 the characteristic diagonal path of the oscillation is plainly visible. It can be traced twice across the channel, beginning on the right wall at the last maximum in the curve, crossing diagonally to the left wall of the downstream tangent, and being reflected back again to the right wall, which it is approaching at the bottom of the picture.

Since the effective cross slope ψ varies from zero at the beginning of curvature to twice the value given by Eq. 32 at the points of maximum on the outer wall, and actually may have a negative slope at the minimum points, it is not surprising that points were found in the South Canal chute that verified Eq. 32. Indeed, as previously stated, this is the equilibrium slope, but it would be disastrous to design the channel freeboard on this basis.

Arthur T. Ippen and the writer[27] have shown that this oscillating disturbance can be eliminated in channels of rectangular cross section by the use of compound curves, in which the central curve, irrespective of its length, is preceded and followed by circular transition curves, each one-half wave length long, having mean radii twice that of the central curve. These transition curves can be thought of as producing counter disturbances of the correct magnitude and phase to interfere with and eliminate the oscillations—first, in the curve itself, and second, in the downstream channel. The same results can also be obtained by transition curves accompanied by banking the channel bottom.

In many cases the use of these concepts and methods will eliminate the need for the model tests recommended by Mr. Hall. However, for complicated constructions involving divided channels and accelerating flow, model studies will certainly be helpful. In such cases the work must be done with extreme care to simulate both depth and velocity very accurately, since the wave pattern, and therefore the acceptability of the design, is so very sensitive to unsynchronized variations of these quantities.

Mechanism of Air Entrainment.—In studying the phenomenon of air entrainment in open-channel flow, the writer has found it convenient to analyze the physical nature of the phenomenon. Therefore, the following remarks are offered in amplification of Mr. Hall's description of the process.

Air entrainment through the free surface of a flowing liquid is essentially a wave phenomenon. More exactly, it is accomplished through the medium of breaking waves, which fold in air and carry it through the interface. The conditions under which waves will break were delineated by H. Helmholz[28] in the nineteenth century. The physical concept behind his analysis is that waves traveling as fast or faster than the relative velocity across the interface (namely, the differential velocity between the water and the air), lose energy, decrease in amplitude, and finally die out. However, waves that travel slower than this differential velocity pick up energy through the interfacial friction, build up their amplitude and finally break, entraining air as they do so. Thus air entrainment is a surface phenomenon and, because of its very nature, is restricted in general to velocities of flow above the critical.

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[28] "Wissenschaflishe Abhandlungen," by H. Helmholz, Leipzig, 1882–1895, Vol. 1, pp. 146–157.

Since the relative densities of air and water are about 1 to 800, a very powerful tendency for the air to separate from the water beginning with the moment of entrainment is always present. The mechanism that carries the air downward against this tendency to separate is the turbulent motion in the liquid. Thus no static equilibrium is possible—only a dynamic equilibrium—and the amount of air in the mixture is determined by the relative value of these opposing forces. There is a direct analogy between this transport of air and sediment transportation, with the air bubbles taking the places of the particles of sand and silt, and the tendency of the bubbles to rise to the surface replacing the tendency of the sediment to fall to the bottom. In the case of sediment transportation, the concentration is far from uniform in the vertical direction, even for laboratory conditions of uniform-sized particles. For coarse materials, little will be found above the bottom layers of the flow. On the other hand, a much more even, but still nonuniform distribution will be found for the transportation of fine silt and clay particles. Much the same conditions will be found in air transport. The depth of penetration of the air below the surface will depend on the size of the bubbles and the level of turbulence. Large bubbles will penetrate less than small ones. The air concentration must be greatest near the surface and may decrease to zero before the bottom is reached if the flow is sufficiently deep.

Since air entrainment occurs by mixing across the interface, help in understanding it can be obtained through the study of other types of interfacial mixing. This phenomenon is greatly exaggerated and presented in "slow motion" for ease in observation in the case of density currents in lakes and reservoirs. A recent review of the general behavior of density currents has been presented by H. S. Bell,[29] with whom the writer has been working for several years. A further discussion of this subject was presented by the writer at the Second Hydraulics Conference in Iowa City, Iowa, June, 1942.[30] Studies of interfacial mixing of density currents in the laboratory show that the mixing is largely restricted to the upper layers of the flow and the bottom layers are relatively undisturbed. Furthermore, the mixing zone soon forms a second definite density flow which travels on the surface of the first and moves at an intermediate velocity between the bottom current and the overlying liquid. This intermediate layer acts as a buffer to inhibit further mixing. It seems very probable that a similar situation exists with air entrainment in open channels. Here the layer of foam is the overlying flow. Since the density of this foam is still very great as compared to air, it would be expected to travel at a velocity much nearer to that of the water than to that of the air. Nevertheless, it is suspected that this velocity is definitely lower than that of the underlying flow.

The preceding considerations lead to numerous conclusions, some of which are outlined in the following items:

1. Because of the nature of the mechanisms of air entrainment and transportation, a nonuniform vertical distribution of air is to be expected. There-

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[29] "Density Currents as Agents for Transporting Sediments," by Hugh Stevens Bell, *Journal of Geology*, Vol. L, No. 5, July–August, 1942, pp. 512–547.

[30] *Proceedings*, Second Hydraulics Conference, Iowa City, Iowa, 1942 (publication pending).

fore, an assumption of uniform mixing is dangerous, although it may be tolerable for high velocities and low depths such as those reported by Mr. Hall.

2. Air entrainment is basically a phenomenon of breaking waves which obtain the energy for breaking from the relative velocity between the air and the water. Therefore, the amount of air entrained probably will be affected by the magnitude and velocity of the wind at the surface of the flow.

3. Waves will not be initiated simply by the high velocity of the flow. Initial disturbances are necessary to cause wave formation. Typical sources of such disturbances are (*a*) channel roughness, which affects the level of turbulence, (*b*) uncompensated curves in channel, (*c*) local irregularities, and (*d*) turbulent structure of wind if velocity is sufficiently high.

4. Neglecting local disturbances, channel roughness should be one of the most important factors in air entrainment and transportation since it determines the turbulence level, which affects not only the amount of surface disturbance but also the depth of penetration of the air.

5. Foam of upper layers probably travels appreciably slower than clear water or layers having lower air concentrations. Therefore, methods of velocity measurements that basically determine velocity of upper layers are probably unreliable and underestimate the true velocity. The color method used in the experiments quoted is subject to this question.

6. Since the entrainment and transportation of air and sediment have so many points in common, it appears that parallel treatments should yield many useful results.

It will be noted that not all of the foregoing conclusions are completely compatible with the assumptions and treatment used by Mr. Hall.

CARL E. KINDSVATER,[31] JUN. AM. SOC. C. E.[31a]—A considerable portion of the paper by Messrs. Kalinske and Robertson is devoted to a study of the hydraulic jump in enclosed conduits. Data obtained from jumps in a sloping pipe verify the application of the pressure-momentum analysis to that phenomenon. In an earlier study[21,32] by the writer, this method of analysis was substantiated for the hydraulic jump in horizontal enclosed conduits. It is interesting to note that the experimental data plotted in Fig. 16 show a slight deviation from the theoretical curve, almost identical with results obtained by the writer. Although this discrepancy is of negligible importance in practical applications, it is of interest to attempt an explanation of the small but definite tendency for the observed "force plus momentum" after the jump to be less than the sum of the initial pressure and momentum. Three explanations were offered by the writer:[21] First, because of the nonuniform distribution of velocities in section (2), the computed momentum, based on an assumption of uniform distribution of the average velocity, is less than the true momentum in the cross section; second, because of the admixture of air with water in section (2), the true velocity of the water is somewhat greater than the average velocity based on the total cross-sectional area of the conduit; and third, because of extremely turbulent, nonuniform flow conditions below the jump, it is impossible to compute accurately the loss of head due to wall friction between sections (1) and (2).

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[31] Asst. Hydr. Engr., Flood Control Section, TVA, Knoxville, Tenn.

[31a] Received by the Secretary February 4, 1943.

[21] "Hydraulic Jump in Enclosed Conduits," by E. W. Lane and C. E. Kindsvater, *Engineering News-Record*, Vol. 121, 1938, pp. 815–817.

[32] "The Hydraulic Jump in Enclosed Conduits," by Carl E. Kindsvater, a thesis presented to the State University of Iowa, Iowa City, Iowa, in 1936, in partial fulfilment of the requirements for the degree of Master of Science.

The first explanation was subsequently shown to be of minor importance by a series of carefully conducted pitot-tube traverses in the region below the jump. It was shown that the nonuniformity of velocities in this section could be expected to account for an increase in M_2 of not more than 2% or 3%.

Now, based on the Kalinske-Robertson study of air entrainment, it is possible to evaluate the second explanation. Fig. 20 shows a comparison of the writer's original results with the same data corrected for the effect of air entrained in the water at section (2). Values of β were read from Fig. 13 and applied to the original data as indicated by Eq. 37. The corrected data show a considerably improved relation to the theoretical curve, but there is still a slight tendency for $F_2 + M_2$ to be somewhat less than $F_1 + M_1$.

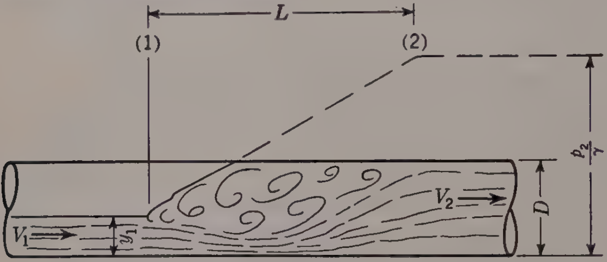

Fig. 20.—Hydraulic Jump in Horizontal Enclosed Conduit

The third explanation for this discrepancy is extremely difficult to evaluate. In the writer's study, $\frac{p_2}{\gamma}$ was corrected for the loss of head due to boundary friction between the section where the pipe first flowed full and the section corresponding to the highest point on the pressure profile. However, this friction loss correction was based on Manning's formula for pipe flow, using measured values of n obtained from tests in a reach of fairly uniform flow conditions. Additional losses in the region of partial-depth flow and miscellaneous losses due to the concentration of velocities along various sections of the pipe are almost impossible to estimate.

It is quite true that the discrepancy shown in Figs. 16 and 21 is of small practical importance. It should be emphasized that the question is largely of academic interest and in no way should invalidate the application of the pressure-momentum analysis to practical problems.

It is gratifying to note the success with which Messrs. Kalinske and Robertson have applied a simplified treatment of the weight term in Eq. 35 for jumps in sloping conduits. In the sentence following Eq. 38, the average length L', used in computing the volume and weight of the jump, is defined as "being approximately equal to the total observed length of the jump, L, minus $\frac{1}{2}(D - y_1)$." Although this definition is sufficiently accurate for jumps having relatively large values of $\frac{p_2}{\gamma}$, apparently it would be quite inaccurate when $\frac{p_2}{\gamma}$ approached the magnitude of D. The length term is especially critical when, as pointed out by the authors, the slope is steep and W becomes a large factor in Eq. 35.

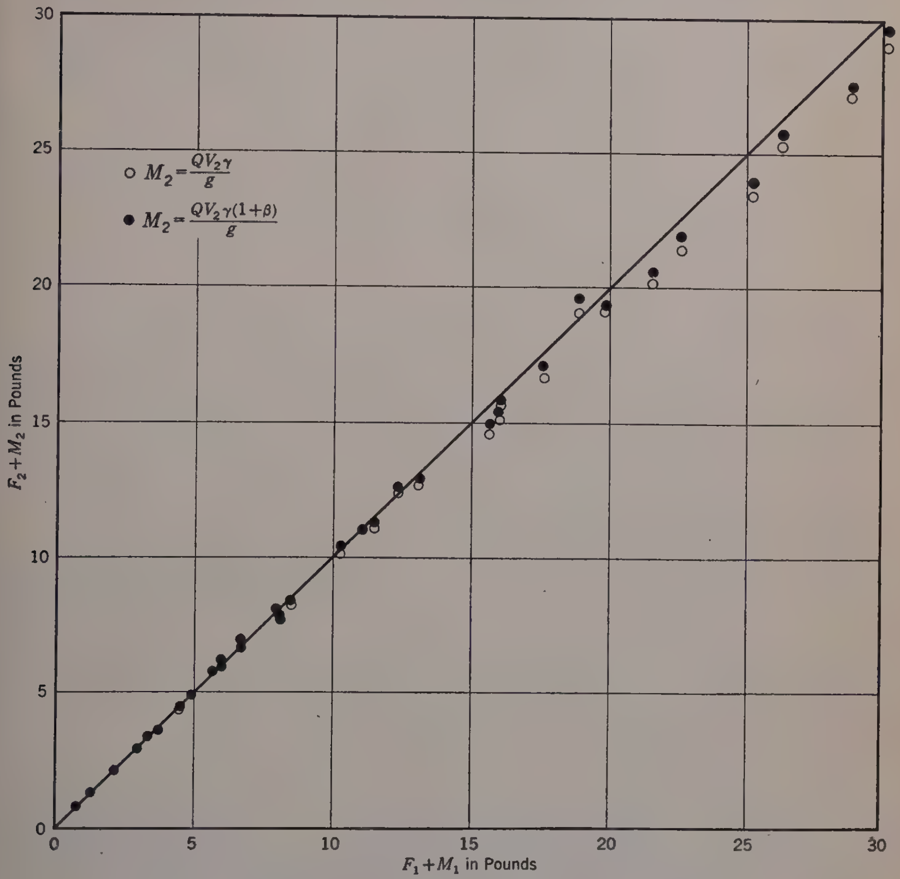

FIG. 21.—VERIFICATION OF EQ. 35 FROM TESTS ON HYDRAULIC JUMP IN HORIZONTAL 6-IN. CIRCULAR PIPE.

Since a practicable treatment for the hydraulic jump in sloping channels, either open or enclosed, is dependent upon measured dimensions of the jump body, the analysis suggested by Messrs. Kalinske and Robertson will have practical significance only when data are available for computing the total length of the jump. Numerous studies[33] on the hydraulic jump in open channels have demonstrated that the length of the jump can be conveniently expressed as a function of Froude's number. A similar relation might have been developed for the jump in sloping enclosed conduits. It is of interest to investigate the approximate form of the latter relation by further reference to the writer's data on the hydraulic jump in a horizontal, circular conduit. Fig. 22 shows a plot of the limited data available, in which the length ratio $\frac{L\,\gamma}{p_2}$ (corresponding to $\frac{L}{d_2}$ for flow in open channels or flow at partial depth in enclosed conduits) is plotted against Froude's number. These data were obtained from tests on three different initial depths—$y_1 = 0.200\ D$; $y_1 = 0.408\ D$; and $y_1 = 0.604\ D$—in a 6-in. circular pipe. Curves drawn through the data are generally similar in shape to length-ratio curves for the hydraulic jump in open rectangular channels. However, apparently due to the circular cross section at section (1), the length ratio varies considerably more with y_1 than with Froude's number. Additional data for this study, for various slopes, initial depths, and Froude's numbers, would be essential for a complete treatment of the hydraulic jump in enclosed conduits.

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[33] "The Hydraulic Jump in Terms of Dynamic Similarity," by Boris A. Bakhmeteff and Arthur E. Matzke, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 630.

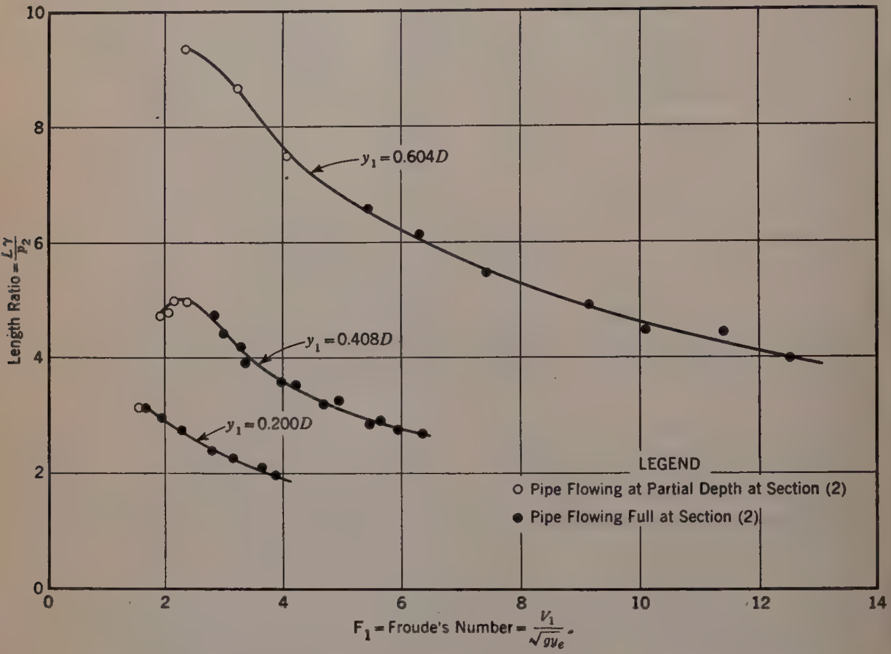

FIG. 22.—LENGTH RATIO, HYDRAULIC JUMP IN HORIZONTAL 6-IN. CIRCULAR PIPES

Corrections for *Transactions:* In September, 1942, *Proceedings*, page 1144, Fig. 12(*c*), change "$\frac{P}{\gamma}$" to "$\frac{p}{\gamma}$"; and, in Figs. 13 and 14, make F_1 bold face ($\mathbf{F}_1$). See also January, 1943, *Proceedings*, page 169, and February, 1943, *Proceedings*, page 308.

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DISCUSSIONS

PRIMARY RÔLE OF METEOROLOGY IN FLOOD FLOW ESTIMATING

Discussion

By Edgar Dow Gilman, M. Am. Soc. C. E.

Edgar Dow Gilman,[3] M. Am. Soc. C. E.[3a]—Much valuable data are contained in this paper, including theories and conclusions on storms and precipitation. Precipitation is undoubtedly one, but only one, of the major factors in the production of floods and is not in itself a controlling factor. Although the subject of precipitation is handled interestingly in this paper, the title, "Flood Estimating," appears to be broader than the material actually covered. For example, the heading of one section is "The Maximum Possible or Ultimate Flood," in which the author sets forth the maximum possible or ultimate precipitation, which does not seem to the writer to be synonymous with floods.

From a study of Ohio River flood records and flood observations in more recent years, there appear to be two factors in floods, regarding which more data and study would be enlightening. The first is the variation in the longitudinal profiles of flood flow and crests. The second is the effect on such profiles of the "timing" of the tributary crests at their outlets relative to the moving crest in the river.

Different floods have quite different longitudinal profiles. When these profiles are plotted as of any instant, at wherever the crest is at that instant, with miles as the horizontal axis and feet elevation as the vertical axis, it is found that the slopes in front of, and behind, crests are quite dissimilar in different floods—some will be gradual and some will be sharp. The lengths of the higher elevations or crests will also vary; some will be short and some will be long. How high a flood is produced in a big river with many tributaries, when there has been a given precipitation over the watershed, depends upon how that volume of water is stored in the moving flood. A given volume in a long, flat crest with gradual slopes will not result in as high a flood as if the same volume were contained in a short, sharp crest with steep slopes. This introduces the factor of "timing"—that is, whether crests in tributaries reach their mouths

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Note.—This paper by Merrill Bernard, M. Am. Soc. C. E., was published in January, 1943, *Proceedings*.

[3] Director, Dept. of Public Utilities, Cincinnati, Ohio.

[3a] Received by the Secretary February 5, 1943.

at such a time as to add to the height or length of the crest in the river or some of each.

In 1913 the Miami River (which enters the Ohio River about 17 miles below Cincinnati) reached its crest on March 26, producing the great Dayton flood. On that date the Ohio River was 4 ft under the flood stage of 52.0 (gage height) at Cincinnati. This caused a pool in the river, and, for a considerable period, water was actually flowing upstream in the Ohio River. In the meantime, a crest in that river was on its way down, but did not reach Cincinnati until April 1, when it crested with a gage height of 69.9, the second greatest recorded flood stage at Cincinnati up to that time (exceeded in 1884 and 1937). The gage height of the Miami River at Hamilton, Ohio, was 34.0 ft at its crest on March 26; and, on April 1, when the Ohio River crested at the mouth of the Miami River, the gage height of the latter was only 4.0 ft. In these six days the Miami River had dropped 30 ft. Presumably the greater portion of the volume of flood discharge from the Miami River added to the length of the flood crest of the Ohio River, rather than to its height, which was probably increased somewhat because of the additional water ahead of the crest. However, the problem is what would have been the effect on this Ohio River crest if the greatest discharge of the Miami River had reached its outlet on March 31, April 1, or April 2, instead of on March 26?

Time obscures details and, in the minds of most people, the 1913 flood in the Miami River and the 1913 flood in the Ohio River at Cincinnati are considered one and the same flood; but the six-day interval between them, with due regard to the flashy character (rapid rise and fall) of floods in this smaller river, probably was a vitally important timing.

That all tributaries to a river would release their maximum discharge at the exact time to produce the greatest increase in height of the crest of the major river as it passed the mouth of each tributary might be equivalent to assuming an improbable, if not an impossible, series of cumulative coincidences. With the maximum precipitation over the watershed of each tributary and such an assumption, a theoretical maximum flood could be shown that at least would be interesting. Although such a drastic eventuality will never be realized, it seems true that floods in major rivers, such as the Ohio, vary in height as more or fewer of the total number of tributaries approach such timings in their discharge; and that, because of variations in the number of such coincidences, it is a practical possibility to have higher floods with other than the maximum precipitations. In other words, unless the precipitation is of very long duration over the entire watershed of a big river, it cannot be stated that the greatest precipitation will necessarily produce the greatest floods.

The writer would suggest again that all river gage heights be read in sea-level elevations. Past records from existing gages could be referenced to the new base and not be lost. Such procedure would add materially to the information to be gained from reading a table of gage heights throughout the length of the Ohio River. It seems unnecessarily confusing to the writer to refer to a gage height of 30 ft in the Miami River at a time when the gage height in the Ohio River is 48 ft, and, at the same time, to realize that water was flowing from the Miami River into the Ohio River.

Corrections for *Transactions:* In January, *Proceedings*, page 107, delete lines 23 to 27, with Fig. 2, and substitute:

"* * * Wakefield, Kans., that the discharge in the Republican River could have greatly exceeded 70,000 cu ft per sec. The maximum flood flow of record had increased from 37,500 to 47,500 cu ft per sec in 1903, and to 70 cu ft per sec in 1915, but for the next twenty years the river failed to reach or exceed the 1915 crest discharge. In May, 1935, the Republican River Basin was visited by a storm which produced a flood peak estimated at 180,000 cu ft per sec. The prevailing maximum-flood discharge, * * *."

On page 110, line 8, substitute "discharge" for "stage."

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

PHYSICAL PROPERTIES THAT AFFECT THE BEHAVIOR OF STRUCTURAL MEMBERS

Discussion

BY JONATHAN JONES, M. AM. SOC. C. E.

JONATHAN JONES,[17] M. AM. SOC. C. E.[17a]—The following discussion of a problem in structural design may be considered a corollary to the author's exposition. In the design of long riveted plate girders, it is frequently economical to construct the flanges of structural silicon steel[18] which, although more expensive per pound, ordinarily permits an increase in unit stress from 18,000 to 24,000 lb per sq in. with a proportionate reduction in section and weight. If, however, a proportionate increase be made in the shearing unit stress for the web plate, it frequently results in a plate undesirably thin as regards either future corrosion or the possibility of buckling. If the shearing unit stress is not increased over that for carbon steel and if a silicon-steel plate is used, an "extra" of some $20 per ton is paid for capacity in shear which cannot be utilized. If carbon steel is retained for the web, it is cheaper, is safe in shear, and is of adequate thickness; but, because of its interaction with the silicon-steel flange, its outermost fibers are inevitably stressed to almost 24,000 lb per sq in., or one third higher than specifications permit.

However, engineers now seem to be in general agreement on the propriety of the evidently economical construction, carbon-web and silicon flanges. As the author has stated, the justification is that not only the unit stress, but also the possibilities of deformation, must be taken into account. In an all-carbon girder, if underdesigning, under future loading, led to an increase of flange unit stress to 33,000 or 36,000 lb per sq in., both web and flange would enter the phase of plastic flow, and the girder would be in danger. In a girder with carbon-web and silicon flanges, however, any future overloading that might bring the flange unit stress to 33,000 or 36,000 lb per sq in. could not cause the web to flow, because the silicon flanges would still be 9,000 to 12,000 lb per sq in. below their yield-point stress of 45,000 lb per sq in., and would still be

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NOTE.—This paper by Wilbur M. Wilson, M. Am. Soc. C. E., was published in December, 1942, *Proceedings*.

[17] Chf. Engr., Fabricated Steel Constr., Bethlehem Steel Co., Bethlehem, Pa.

[17a] Received by the Secretary January 14, 1943.

[18] Specifications, American Society for Testing Materials, Designation, A.S.T.M.-A94.

elongating under practically their original modulus. The carbon web is locked in a high-yield-point flange that will not let it flow, and without flowing it cannot fail, being a material of high ductility.

The author refers to the eyebar (evidently the forged-head bar of several generations' standing) as an advantageous form from the standpoint of energy absorption, the advantage arising from the lack of any spots or localities of reduced area. Attention might here be called to a description[19] of a different type of tension-resisting bar, in which reinforcing-pin plates were welded on to each end of a long flat plate. In the case described,[19] this bar proved more economical of manufacture than a forged-head bar, and it obviously possesses the characteristic praised by the author of having no point of reduced cross section from end to end.

The last two sentences of the "Synopsis" open up a wider field than that of which the author disposes. The vital question of correlation between uses to which a material is to be put, and tests specified for its acceptance, is suggested by the "Synopsis" but not conclusively treated in the paper. Tests for strength and ductility are shown to be appropriate in specifications for structural steel, but does the author answer his final question of "whether or not there are other characteristics of the material that should be considered"?

For structural carbon steel, it may be stated safely that there are no other characteristics, unless and until new types of application must be considered; if, for instance, welding is contemplated, a test that would check directly on "weldability" might be wanted. In general, however, considering riveted bridges, buildings, and similar structures, years of experience and millions of tests have convinced the engineer that if structural carbon steel passes the routine tests for strength and ductility, it may be considered also to possess all of that dependability under adverse conditions which the engineer has learned to expect of it.

The danger lies in carrying the same reasoning, by analogy, over into the specifying of tests for harder steels and for other materials, where no such background of experience may yet exist. Carbon steel, with 60,000 lb per sq in. tensile strength and, say, 26% elongation in 8 in., can safely be subjected to 18,000 lb per sq in. average stress combined with some form of coped or otherwise suddenly altered cross section. Can some harder steel, with 80,000 lb per sq in. and, say, 18% elongation in 8 in., be used safely in the same notched form at some higher unit stress? Often not. Often, it would not safely carry the same unit stress as the carbon steel. A new requirement has been introduced by the sudden change of cross section—accommodation to point concentrations of stress. Will the routine tension and bend tests establish that such a power of accommodation exists? They will not.

Supplementary requirements are not always overlooked. Sometimes for a harder structural steel, a "drifting" test has been imposed, at least on occasional samples, if not in a routine way. In many instances, however, the end use and the acceptance test seem to be further apart than is logical. (In a "drifting" test, punched holes in a narrow bar with sheared edges are considerably enlarged by driving tapered pins, the measure of acceptability being the degree of enlargement before fracture at edge of hole or edge of bar.)

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[19] "Test of Model of the Tacoma Narrows Anchorage Bar," by Bruce Johnston and H. J. Godfrey, *Welding Journal*, Welding Research Supplement, August, 1939, p. 253s.

The end use of bridge cable wire is to withstand tension combined with a highly overstraining cold bend around a small-radius sheave at the anchorage, and also to withstand tension combined with heavy cross loading in cable bands and saddles. That the customary tensile and bend tests are not prima facie evidence of this characteristic was proved by the serious failure, in service, of wire that had passed such tests with ease, but that, by an innovation in the process of manufacture, had been rendered vulnerable to the critical conditions just named.

The end use of a bronze bushing or bearing plate is to withstand rubbing on steel under heavy pressure without excessive wear, and to withstand this without seizing or scoring despite considerable fault in the lubrication; but the specified acceptance tests cover sometimes compressive deformation and sometimes tensile strength and elongation.

The end use of arc-welding filler metal is to make a sound, strong, and ductile joint with the material to be welded, under a wide range of variables such as position and temperature; but the specified physical tests are made upon the filler metal as laid in and upon itself, under the most uniform and ideal conditions of deposition.

None of the situations just described need necessarily to be dangerous, but the frame of mind they imply is so. Danger is absent if and when a sufficient body of experience has been built up, to prove that if the acceptance tests are passed, none of the truly important properties, even though the tests do not challenge them, can be absent. Prudence dictates a greater effort than is always in evidence, to make the acceptance tests fit the critical aspects of the end use; and the fact should be squarely faced that in many instances an increased cost of testing will have to be absorbed by the user.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

APPLICATION OF SOIL MECHANICS IN DESIGNING BUILDING FOUNDATIONS

Discussion

BY MESSRS. JACOB FELD, AND LEONARD ZEEVAERT

JACOB FELD,[15] M. AM. SOC. C. E.[15a]—The use and the limitations of soil tests and soil theories that should be considered in building foundation design are summarized in this paper. Definite rules are set forth for cohesionless soils, and the general advice is given that the "floating method" be used in cohesive soils.

The thirteen conclusions and recommendations listed for cohesionless soils (see heading: "Introduction—Stress-Deformation Characteristics of Cohesionless Soils") are open to question since, in some instances, they do not agree with either experiment or experiences. For example:

(*a*) There is no justification for the assumption that the intensity of pressure at the edge of a rigid area is zero. The experimental works of W. S. Housel,[16] M. Am. Soc. C. E., of R. B. Fehr and C. R. Thomas,[17] and of Frederick J. Converse,[18] Assoc. M. Am. Soc. C. E., definitely show the existence of some pressure along the edges of footings laid in sand.

(*b*) The uniformity of base pressure in pier foundations is the result of the rigidity of a deep footing rather than of the depth of the footing. Similar uniformity of base pressure should be expected under a gravity type of abutment built in a shallow excavation.

(*c*) A rewording of this rule might clarify its meaning, to the effect that the unit bearing is independent of the width of the loaded area. However, this is not consistent with conclusion (*i*). The effect of footing shape should be considered, as well as the corresponding effect of edge resistance in sustaining loads.

NOTE.—This paper by A. Casagrande, Assoc. M. Am. Soc. C. E., and R. E. Fadum, Jun. Am. Soc. C. E., was published in November, 1942, *Proceedings*.

[15] Cons. Engr., New York, N. Y.

[15a] Received by the Secretary February 6, 1943.

[16] *Research Bulletin No. 13*, Univ. of Michigan, Ann Arbor, Mich., 1929.

[17] *Bulletins*, Pennsylvania State College, State College, Pa., 1912–1915.

[18] *Civil Engineering*, April, 1933, p. 207.

(*d*) This rule is not substantiated by tests of bearing values on deep caissons. The more likely rule should read:

"The increase in ultimate bearing capacity of a cohesionless soil with depth is approximately as the square of the depth of the loaded area."

(*e*) and (*f*) The author suggests correctly that, taking depth into account, different allowable bearing values should be assumed for interior and exterior wall footings.

(*g*) The elimination of the Boussinesq formula as a tool to indicate the range of expected values is not wise until a more accurate method is devised. Although the Boussinesq formula is based on the assumption of isotropic elastic materials (which assumption does not include soils at failure), it should be emphasized that the theory of elasticity of materials is similarly limited. Since, under small loadings, the strains in soils are substantially homogeneous and the lack of tensile strength does not come into play until the tensile components of internal shear strains exceed the initial compression, there is a range of loading for which the Boussinesq formula, in the original or modified form, does serve a useful purpose.

In the list of recommendations for noncohesive soils, at least a warning of the effect of unwatering on settlements also should be included.

The statement that the action of foundations in clays can be properly predicted only if the loads are placed in excavations deep enough to "float" the building, is wise. Mention might be made of the practice in clay localities (Texas, Connecticut, etc.) of not removing the last foot or so of the excavation until just before placing the concrete so as to reduce the settlement from recompression of the clay swelling at subgrade.

The time-settlement records in Fig. 15 for building IV show a differential settlement of the same amount as the maximum settlement and for building V, a differential settlement of 13 in., with a maximum settlement of only 10.5 in. This indicates that some part of building IV did not settle at all, and part of building V rose at least 2.5 in. The possibility of such behavior when parts of a building (or adjacent buildings) are of different weights is not often considered or expected.

Much can be learned of foundation design by the analysis of corrective measures for buildings showing differential settlements. The writer described a few cases in a paper in 1930.[19] A list of references on the subject of bearing capacity published before 1913, in readily available literature as determined for the design of structures now in use, is as follows:

(1) **"Preliminary Foundation Tests for the St. Paul Building,"** *Engineering Record*, Vol. 33, May 2, 1896, p. 388.

(2) **"Safe Load on Soil at New Orleans, La.,"** *Engineering News*, Vol. 41, May 11, 1898, p. 330 (correction, p. 333).

(3) **"Foundation Construction for the New Capitol for South Dakota,"** by Samuel H. Lea, *Engineering Record*, Vol. 57, April 4, 1908, p. 437.

(4) **"Bearing Tests for Heavy Foundation Loads,"** *Engineering Record*, Vol. 60, July 10, 1909, p. 55.

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[19] "Several Foundation Repair Jobs," by Jacob Feld, *Proceedings*, Brooklyn Engrs. Club, April, 1930.

(5) **"Testing Bearing Power of Hard-Pan. Extension, Whitehall Building, New York City,"** *Engineering Record*, Vol. 61, June 18, 1910, p. 792.

(6) **"Tests and Costs of Making a Test of the Bearing Power of Soil for a Building,"** *Engineering-Contracting*, Vol. 34, July 13, 1910, p. 31.

(7) **"Test of Bearing Capacity of Sand under Municipal Building, New York City,"** *Engineering Record*, Vol. 62, July 9, 1910, p. 46; July 16, 1910, p. 57; and *Engineering-News*, Vol. 63, January 6, 1910, p. 24; Vol. 64, November 17, 1910, p. 525.

(8) **"Device for Making Sub-Surface Tests of the Bearing Power of Soils with Some Examples of Operation,"** *Engineering-Contracting*, Vol. 34, August 3, 1910, p. 94.

(9) **"Testing Soil Below the Surface for Foundation Loads,"** *Engineering Record*, Vol. 62, July 16, 1910, p. 71; Vol. 63, May 6, 1911, p. 512.

(10) **"Testing Foundations at the Municipal Building, New York,"** *Engineering Record*, Vol. 63, February 18, 1911, p. 196.

(11) **"Standard Test of Soil,"** by Rudolph P. Miller, *Engineering Record*, Vol. 66, July 27, 1912, p. 112.

(12) **"Soil-Bearing Tests,"** *Engineering Record*, Vol. 66, September 14, 1912, p. 304.

(13) **"Hard-Pan and Other Soil Tests,"** by J. Norman Jensen, *Engineering News*, Vol. 69, March 6, 1913, p. 460 (also editorial, p. 463).

(14) **"Building Foundations,"** by J. A. Smith, *Journal*, Associated Eng. Soc., Vol. 36, April, 1906, p. 155.

(15) **"Results of Tests on Chicago Hard-Pan at a Depth of 57 Feet Below Lake Level,"** by Frank A. Randall, *Engineering-Contracting*, Vol. 37, April 17, 1912, p. 436.

(16) **"Supporting Power of Soils,"** by Randall Hunt, *Journal*, Associated Eng. Soc., Vol. 7, June, 1888, p. 189; and *Engineering News*, Vol. 19, June 16, 1888, p. 484.

(17) **"Construction of the Buildings, Bridges, Piers and Docks at Jackson Park,"** *Engineering Record*, Vol. 28, August 26, 1893, p. 199.

(18) **"Allowable Pressure on Deep Foundations,"** by Elmer L. Corthell, *Engineering News*, Vol. 56, December 20, 1906, p. 657; and *Engineering Record*, Vol. 54, December 15, 1906, p. 647 (editorial).

(19) **"Foundation Pressure on Hard-Pan; Proposed Rule,"** by Rudolph P. Miller, *Engineering News*, Vol. 74, December 29, 1910, p. 727; and *Engineering Record*, Vol. 62, December 31, 1910, p. 783.

(20) **"Sand Foundations for High Buildings,"** *Engineering Record*, Vol. 66, September 21, 1912, p. 310.

(21) **"Report on Unit Pressure Allowable on Road-Beds of Different Materials,"** *Proceedings*, A.R.E.A., Vol. 13, 1912, p. 388.

(22) **"Failure of the Transcona Grain Elevator,"** *Engineering News*, Vol. 70, November 6, 1913, p. 944.

Leonard Zeevaert,[20] Jun. Am. Soc. C. E.[20a]—The material contained in the paper is of great value and very encouraging for it shows the ready application of soil mechanics in the design of building foundations. The writer has had the opportunity to design the foundations for three buildings constructed in Mexico City, Mexico, in which principles similar to the ones stated in the paper have been used.

In each of these buildings the basement was excavated to variable depths to give a more uniform and favorable distribution of loading and thus to decrease the dishing effect. Early indications show greatly decreased dishing as compared to similar adjacent buildings and to other buildings in the city which, according to usual practice, have equal depth of excavation throughout the foundation area. Decreased average settlements are obtained because of the more uniform stresses within the clay strata. The writer found that the idea of excavating to variable depths in order to create more uniform stresses in the compressible strata, and therefore to reduce differential settlements, helps considerably and is readily applicable to very compressible soils, such as Mexico City clays. He has called this type of foundation a "compensated foundation."

To compute the load distribution on the foundation slab when a rigid foundation is to be used in connection with underlying clay, "the edge load problem" was devised. The compensated excavation and an edge load applied along the boundary of the foundation are the means used to obtain an essentially uniform stress in the clay stratum below the structure. The load thus applied at the surface to hold these uniform stresses within the clay stratum is the load used to compute the foundation structure of the building in question.

One example of the aforementioned items is the Wong Office Building (165 by 56 ft and 105 ft high) under which there is a stiff fill underlain by a very soft clay stratum extending about 90 ft to the underlying sand stratum. The building is of concrete with an average net load of 0.9 ton per sq ft. In this case, the ratio of the load of the building to the weight of the soil excavated was about 9 to 5. In the analysis it was recognized that adjacent buildings transmitted considerable stress to the clay beneath the Wong Building and these stresses were taken into account in the settlement analysis.

The solution of the edge load problem is explained briefly by reference to Figs. 24(*a*) to 24(*e*). Fig. 24(*a*) shows the stress distribution under the foundation and strata due to a flexible foundation on clay, and Fig. 24(*b*) shows the same for a rigid foundation. Therefore, in a foundation that may be considered rigid, the stress distribution in the clay stratum and the load distribution on the surface must be approached as demonstrated in Fig. 24(*b*). By "compensating" the excavation, the first step to this approach is obtained. Fig. 24(*c*) shows the improved distribution of stress in the clay strata. However, there will be a drop in the stress distribution within the clay strata (Fig. 24(*c*)) at the ends corresponding to the boundary of the excavation. Therefore, unless the foundation of the building is sufficiently rigid, dishing the foundation structure is possible. By applying edge loads as shown in Fig. 24(*d*), a fairly

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[20] Prof., Soil Mechanics, School of Civ. Eng., Univ. of Mexico; Cons. Engr., Mexico City, Mexico.

[20a] Received by the Secretary February 11, 1943.

uniform stress to the very edge of the foundation and, at the same time, a load distribution on the surface that approaches the distribution for a rigid foundation (Fig. 24(*e*)) may be obtained. This load can be supplied by designing the foundation sufficiently rigid.

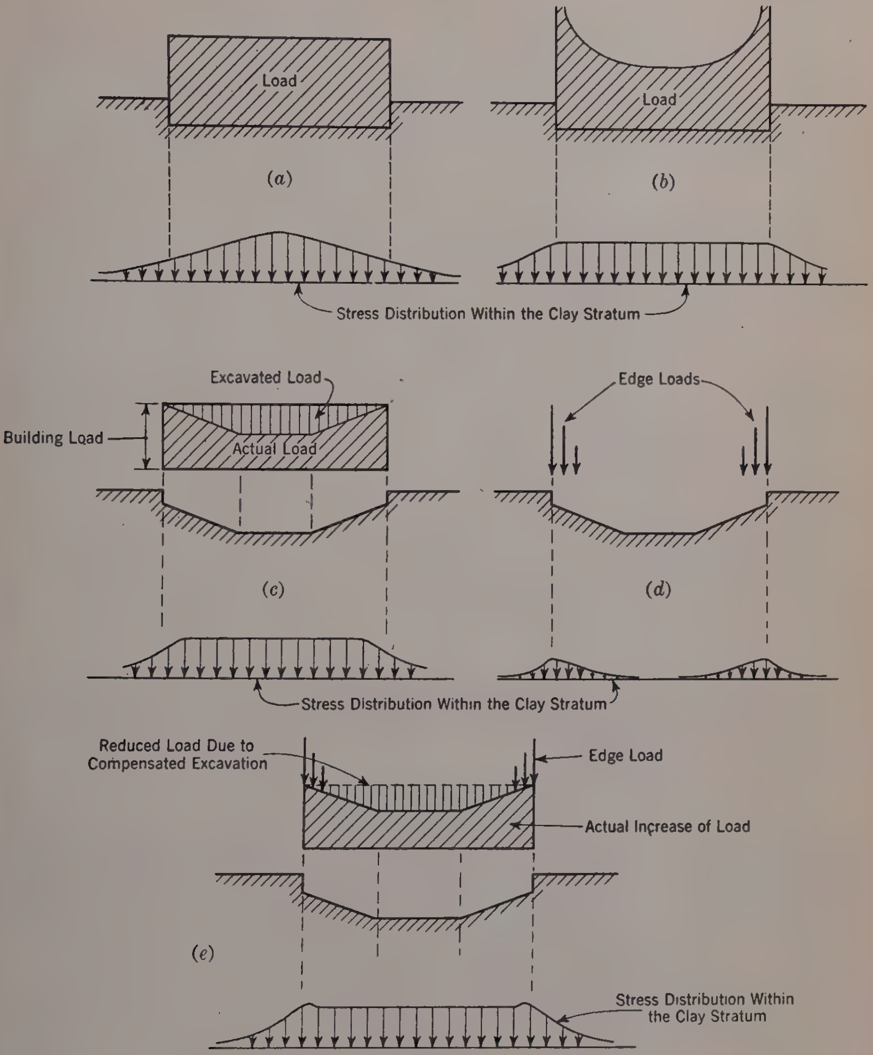
Load
(*a*)
Load
(*b*)
Stress Distribution Within the Clay Stratum
Excavated Load
Building Load
Actual Load
(*c*)
Edge Loads
(*d*)
Stress Distribution Within the Clay Stratum
Reduced Load Due to Compensated Excavation
Edge Load
Actual Increase of Load
(*e*)
Stress Distribution Within the Clay Stratum

FIG. 24.

This building indicated an average settlement of 1.2 in. in nine months after the excavation load was reached, and, three months later, a total settlement of 1.6 in. after application of practically all load. The differential settlements were of the order of 0.08 in. (maximum) at points of about 50 ft apart. The settlement analysis indicated 2 in. at the end of the construction period and about 16 in. per decade after application of the load. Early in the construction period the settlements were 0.8 in. greater at one end, because this end during the construction period was more heavily loaded than the remainder of the building, but the tilting of the foundation did not exceed 0.8 in. at the completion of the loading period. Differential settlements remained under 0.08 in., and no dishing effect has been noticed. The excavation depths were not appreciably greater than those commonly used in Mexico City. A regular concrete foundation with beams and 10-in. thick slabs was used, and no cracks have developed. This is of particular significance as, in Mexico City, most buildings with fewer stories have shown visible cracks after the construction period. The writer has used his theory of foundation computations in two other buildings with similar results.

The Boussinesq formula derived from the theory of elasticity was used for stress estimates. Predicted settlements have been of the order of 30% greater than observed settlements. The void ratio of clays encountered at these sites is of the order of 9, and compressibility coefficients at the loads usually encountered are of the order of 1 cm^2 per kg. Samples were taken from the top, bottom, and middle of the clay stratum and tested in the writer's private laboratory. The coefficients used were the average weighted values of the data obtained from the three depths.

Concerning the heaving of the foundation, the writer[21] has found that when the excavation extended through the stiff fill underlying the clay, heaving became very noticeable but ceased after a short period. Therefore, the opinion of the writer is that when excavations occur in very soft clay the heaving is mainly in the nature of lateral plastic flow into the excavation rather than an expansion of the underlying clay.

Corrections for *Transactions*: In November, 1942, *Proceedings*, page 1498, line 13, change "$\Delta e''$" to "Δ_1"; on page 1499, lines 1 and 2, change "$\Delta e'$" and "$\Delta e''$" to "Δ_2" and "Δ_1," respectively; in Fig. 15(*b*), p. 1504, replot Curves V as shown in corrected Fig. 15.

FIG. 15 ON OPPOSITE PAGE.

On page 1516, line 20, change "elevations" to "movements"; and in Fig. 22, change "141" tons per sq ft and "0.39" ton per sq ft to "1.41" and "−0.39," respectively.

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[21] "Aplicación de la Mecánica de los Suelos al Estudio de las Excavaciones en Arcillas Saturadas," by L. Zeevaert, *Ingenieria y Arquitectura*, Agosto, 1942.

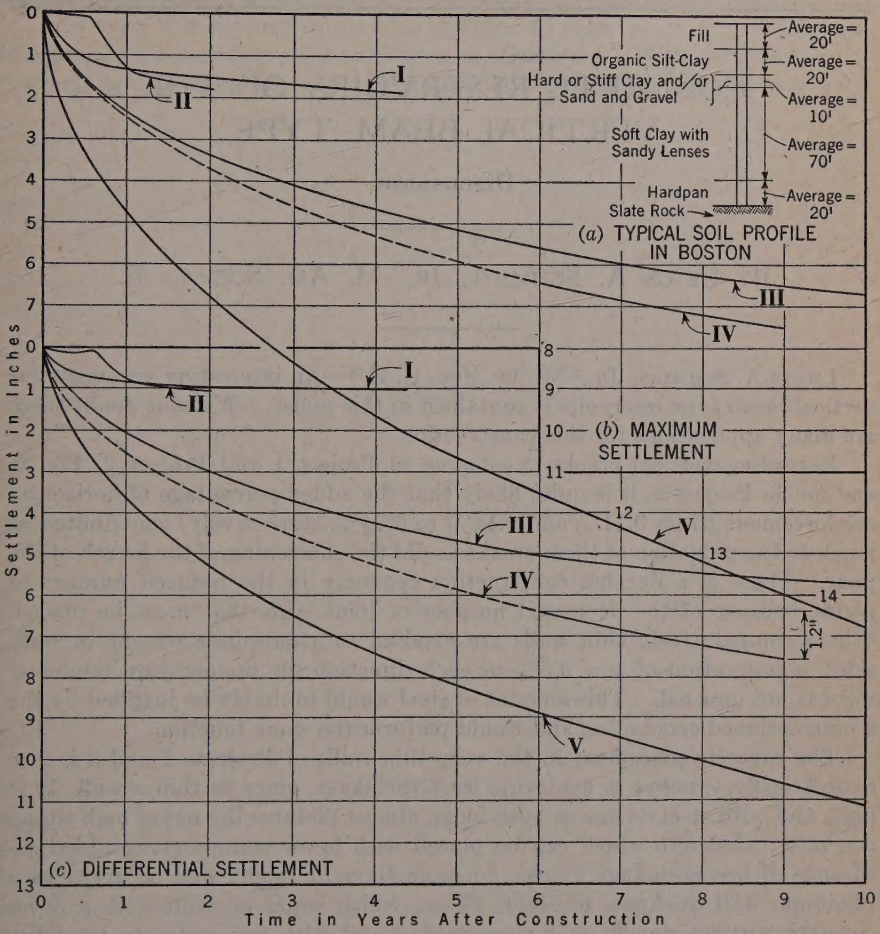

FIG. 15.—TIME-SETTLEMENT RECORDS, BOSTON, MASS.

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CONCRETE RESERVOIRS OF THE VERTICAL-BEAM TYPE

Discussion

BY LEWIS A. SCHMIDT, JR., M. AM. SOC. C. E.

LEWIS A. SCHMIDT, JR.,[2] M. AM. SOC. C. E.[2a]—An interesting argument for vertical-beam type reservoirs is contained in this paper. Without doubt there are many applications for this construction.

Regarding vertical cracks developing in Project 1 and Project 2, Fig. 5, and not in Project 3, it is quite likely that the added percentage of horizontal reinforcement (from 0.11% and 0.14% to 0.34%, respectively) contributed as much to the reduction of these cracks as did the shortening of the length of the pour. There is a decided construction economy in the reduced number of pours, because of the decreased number of joint seals that must be placed. Where comparatively thin walls are exposed to atmosphere on one or both sides, a proportion of $p = 0.5\%$ in each direction for temperature reinforcement is not unusual. This amount of steel would probably be justified by the aforementioned economies, and would perform the same function.

The concrete placement in the very thin walls of Projects 2 and 3 is also diametrically opposed to achieving least shrinkage, since so thin a wall, 10 ft high, and with steel on one or both faces, almost dictates the use of high slump concrete. Concrete which can be placed with lower slumps gives added assurance of less shrinkage cracks, so some thought might well be given to a minimum wall thickness of about 12 in., which could be built with a dryer concrete without danger of honeycombing and with less water to be dehydrated. It seems probable that the disparity of roof thicknesses between Projects 1 and 2 contributed to the cracking in Project 1 as much as the restraint offered by the rigid connection.

The author mentions that conventional copper seals have not proved to be satisfactory. These have been used so often in heavy construction, particularly in conjunction with premolded joints, that it would be desirable to set forth their deficiencies in the reservoirs.

NOTE.—This paper by C. Maxwell Stanley, Jr., M. Am. Soc. C. E., was published in December, 1942, *Proceedings*.

[2] Acting Constr. Engr., TVA, Hales Bar Dam, Chattanooga, Tenn.

[2a] Received by the Secretary January 11, 1943.

Mr. Stanley advisedly calls attention to limitations of the design, and undoubtedly the seventh one would be very important in this case, especially where the bottom slab rests on the footing as well as on the foundation. Differential pressures at this point due to the wall load could contribute some unequal settlement and cracking, particularly of the bottom slab.

This type of construction may be expected to show up to best advantage in square or rectangular rather than circular structures. Within its limitations it would be interesting to see a comparative cost analysis of various designs.